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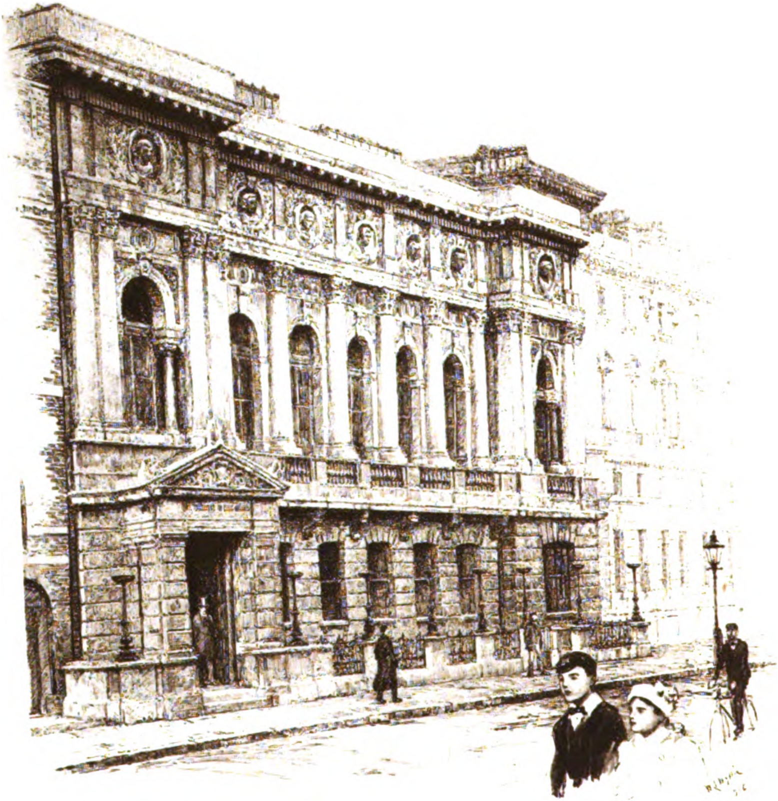

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THE INSTITUTION OF CIVIL ENGINEERS.

JULY, 1910.

MINUTES OF PROCEEDINGS

OF

THE INSTITUTION

OF

CIVIL ENGINEERS;

WITH OTHER

SELECTED AND ABSTRACTED PAPERS.

VOL. CLXXXII.

EDITED BY

J. H. T. TUDSBERY, D.Sc., M. INST. C.E., SECRETARY.

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THE INSTITUTION OF CIVIL ENGINEERS.

SESSION 1909-1910.—PART IV.

Sect. I.—MINUTES OF PROCEEDINGS.

---

5 April, 1910.

ROBERT ELLIOTT-COOPER, Vice-President,
in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Rustat Blake, B.A. (*Cantab.*)
Hubert Cecil Booth.
Samuel Christopher Chapman.
Henry Herbert Crabtree.
Sir Robert Harvey.
Thomas Charles Pulman.
William Charles Ernest Smith.
Theodore Stevens.
Edward Arthur Wilson.

And had admitted as

Students.

Ernest Ainsworth.
Ernest Daniel Alexander.
George Joseph Allan.
Alexander Keith Andrew.
Hugh Gordon Bambridge.
Thomas Henry Maughan Barnett.
John Barratt, Jun.
Wilfrid Philip Barron.
Harold Edgar Greaves Beevers.
Oswald Irving Bell.
John Victor Bessant.
James Main Linton Bogle.
Ernest Stanley Bradley.
James Anderson Burnett.
John Lawrence Butler.
Charles Graham Campbell.
Herbert Cawthra.
Charles Francis Chute.
George Stanley Clarke.
Wilfrid Randall Clarke.
Charles Arthur Coard.
Robert Charles Crawley.
Herbrand Leo Crosby.
Herbert Victor Dank.
Thomas Alexander Davies.
Tudor Huab Davies.
Horace Cardwell Day.
Gerald Antony Delaney.
Jan Dickman.
Arthur Pearse Dicksee.
Randolph Edward Docwra.
Richard Brooke Dorman.
Archibald Eastmond.
Joseph Arthur Ellis.
William Evers.
John Burton Fallowfield.
Maurice Farnell.
Herbert Felton.
Eric Harold Fforde.
Alexander Ross Fyfe.
John Walker Gardner.
Harry Frederic Higgins Gibson.
William Griffin Glenister.
Charles Reginald Green.
John Francis Haigh.
William Gathorne Hardy.

Students—continued.

Arthur Hargreaves.
Stanley Muirhead Haycraft.
John de Vars Hazard.
Frederic John Lockwood Heward.
Otto William Hodsman.
Duncan Hook.
Arthur Cecil Howarth.
David Arnold Howell.
Edward Cecil Ingleby.
Harry Robertson Jack.
Philip Henry Reiss Jephson, B.A. (*Cantab.*)
Leslie Charles Kemp.
Edward Robert Kendrick.
Otway Prior Kennedy.
Andrew Kilgour.
George Whitaker King, B.E. (*New Zealand*).
Alan Wilfrid Ladner.
Frederick Tremayne Lane.
Francis de Lautour.
David John Leacock.
Percival Arthur Rupert Leith.
Rupert Arthur Percival Littlewood.
John Ireland Long.
Herbert Edgar Lunn.
George Nicol Lyall.
Ian Stuart MacKay.
William Rowand McKim.
Richard Maclean.
Hugh Robert Munro Macmillan.
John Wyld Mather.
Ivor Edwin Mercer.
Denis Laurence Monaghan.
Harry Morris.
James Thomson Morrison, Jun.
Allan Langton Myers.
Henry Newton.
Stephen Wallace John Norris.
Neville Marriott North.
Stephen Alexander Orr.
Hugh Logan Parker.
Stanley Sherman Parkes.
William Pearce.
Barwise Percival.
Hubert Cyril Phillips.
David Barnes Pryde.
Alexander Bruce Purves.
Frederick Charles Leslie Racker.
William Henry Rean.
William Owen Ridley.
Leonard Mould Robinson.
Edward William Robison.
Gilbert Dalgleish Ross.
William Butler Ruddock.
Cyril Eustace Sayer, B.Sc. (Engineering) (*Lond.*).
George William Scriven.
George Reid Shaw.
Percy White Smith.
Victor Robert Richmond Smith.
Sidney George Stanton.
Bernard Stapleton.
George Stewart.
Frederick William Scott Storry, B. Eng. (*Sheffield*).
Charles Norman Stuart.
Ronald Summerscale.
William Lennox Tod.
John Gerard Towlson.
Edmund George Townshend.
Charles Harold Traill.
Leonard Cecil Turner.
Roland King Uhthoff, B.A. (*Cantab.*)
Mervyn Stanley Vernal.
Hubert Edmund Walker.
Norman Lyell Wallis.
Douglas Wallwork.
Edgar Longshaw Walton.
John Gutteridge Ward.
John Patchell Watson.
John Wintringham White.
Harry Hood Whyte.
Tom Herbert Williams.
Allan Wilson.
Charles Arthur Wilson.
Robert Kendal Wortley.
Thomas Forbes Young.
Douglas Stanley Zossenheim.

The Scrutineers reported that the following Candidates had been duly elected as

A Member.

William Cleaver.

Associate Members.

Henry Charlton Amos.
Clifford Oakes Bower.
Alfred William Brayshay.
Harry Bertram Edwin Brown.
Walter Henry Budgett.
George Edwin Bohun Coulcher.
William Nicholas Gerald Douglass, B.A.I. (*Dubl.*)
Hubert Dyson.
James Donald Eadie.
Bernard Pemberton Ellis.
Harold Fortescue Flannery.
Donald Corbet Fletcher.
George Purdon Geen.
Charles Glenday.
Frederick Brokenshire Goddard.
Ivor Ellis Gooch.
Laurence Thornewaite Grace.
Bernard Booth Granger.
George Albert Grimoldby.
John Herbert Haiste.
Harold Moffat Swire Haughton, B.Sc. (*McGill*), Stud. Inst. C.E.
David Henderson.
George Walls Hilditch.
John Arthur Hoyle.
Herbert Richard Kiernan.
Herbert Edward King, Stud. Inst. C.E.
Luke Hamilton Larmuth.
Ernest Latham, Stud. Inst. C.E.
Leopold Leighton.
Alfred Edwin Loach.
James McLellan, B.Sc. (*Victoria*).
Nathaniel Martin, B.Sc. (*Glas.*), Stud. Inst. C.E.
Cyril Middleton.
Tom Lord Norfolk.
Alfred George Benjamin Palmer.
Harry Leonard Percy.
Ernest Lancelot Pledger.
Ernest William Porter.
Herbert William Rattray.
Frank James Redman.
Francis Aymer Richardson.
John Stocks Richardson, M.Sc. (*Leeds*), Stud. Inst. C.E.
Harold William Townend Churchill Sabine, Stud. Inst. C.E.
Sidney Marshall Senior.
Leonard Mountford Smith.
William Wycliffe Spooner, B.A. (*Cantab.*)
Edward John Stead.
Harford George Stephens.
William Sutcliffe.
William Norman Thomas, B.Sc. (*Lond.*)
Alan Chichester Thomson.
George William Arthur Trimmer.
Harold George Turner.
Sydney Upton, Stud. Inst. C.E.
John Henry West.
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(*Paper No. 3764.*)

"The New Clyde Bridge of the Caledonian Railway at Glasgow."

By Donald Alexander Matheson, M. Inst. C.E.

The New Clyde Bridge, which was completed in 1905, is perhaps the most outstanding structural feature of the recent extension of the Caledonian Railway Central Station in Glasgow, the scheme and general arrangement of which have already been described to The Institution.[1] The new bridge, which carries the widened railway across the River Clyde in the immediate precincts of the station (*Fig. 1*), is within a stone's throw of Telford's reconstructed "Glasgow Bridge," and immediately west of the railway-bridge which was built in connection with the old Central Station.[2]

General Description.

For structural reasons it was considered expedient to have the new bridge and the old bridge somewhat apart at the middle, but they are close together at the ends.

The general form and dimensions of the new bridge are shown by Figs. 2–5, Plate 1, and Figs. 21 and 22, Plate 2, from which it will be seen that the structure is in the nature of a great platform of steelwork carrying the main part of the Central Station yard, and the approach-lines thereto, across the river and the street on either side of it. The width between the parapet-girders is 114 feet at the middle of the centre span, fanning out to 205 feet at the north end, and to 118 feet at the south end; and it is believed that there are few wider railway-bridges in existence. The total length of the five spans, measured on the centre-line, between the faces of the abutments, is 702 feet 6 inches.

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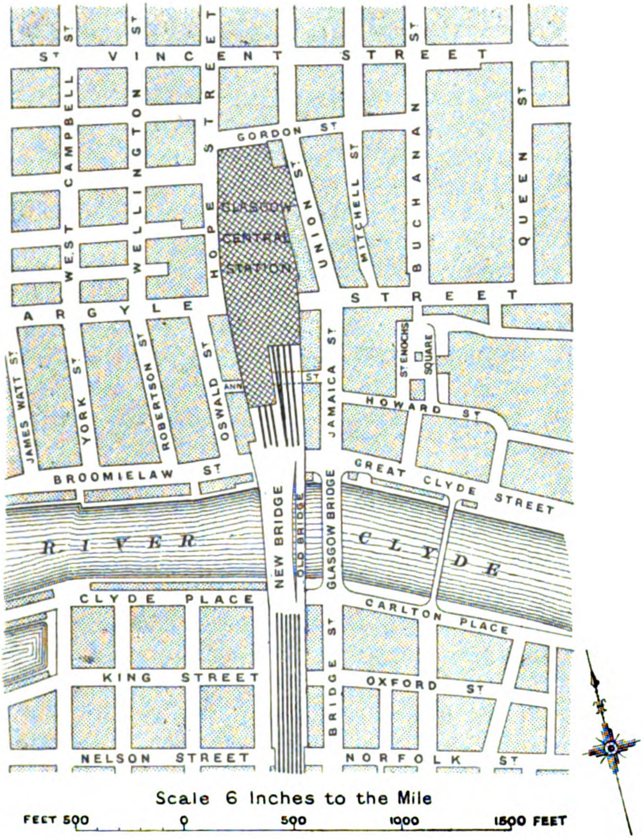
[1] D. A. Matheson, "Glasgow Central Station Extension." Minutes of Proceedings Inst. C.E., vol. clxxv, p. 30.

[2] B. Hall Blyth, "The Caledonian Railway Viaduct over the River Clyde at Glasgow," Minutes of Proceedings Inst. C.E., vol. lxi, p. 190.

Fig. 1.


ST VINCENT STREET
WEST CAMPBELL ST
WELLINGTON ST
HOPE STREET
GORDON ST
UNION ST
MITCHELL ST
BUCHANAN ST
QUEEN ST
GLASGOW CENTRAL STATION
ARGYLE STREET
JAMES WATT ST
YORK ST
ROBERTSON ST
OSWALD ST
ANN ST
JAMAICA ST
ST ENOCHS SQUARE
HOWARD ST
BROOMIELAW ST
GREAT CLYDE STREET
RIVER CLYDE
NEW BRIDGE
OLD BRIDGE
GLASGOW BRIDGE
CLYDE PLACE
CARLTON PLACE
KING STREET
BRIDGE ST
OXFORD ST
NELSON STREET
NORFOLK ST
Scale 6 Inches to the Mile
FEET 500
0
500
1000
1500 FEET

PLAN OF SITE.

The bottom of the foundation of the southernmost river pier is about 110 feet below rail-level, and as the top of the pilaster is about 30 feet above rail-level, the total depth from the top of the pilaster to the bottom of the foundation is about 140 feet. The new bridge carries nine lines of rails at its north end, which converge to eight at its south end; while the old bridge carries four lines.

The range of tide in the River Clyde at the site of the bridge is about 11 feet, and high-water level is taken to be 6·20 feet above Ordnance datum.

The substructure of the bridge consists of masonry abutments and piers, the latter being composed of a series of cylindrical pillars (Figs. 3–6, Plate 1). The foundations of the abutments are only about 15 feet and 9 feet below the surface of the streets on the north and the south side of the river respectively, but the piers are carried down to considerable depths, parts of their foundations being as low as 70 feet below high-water level. The abutments and piers of the new bridge were fixed by statute to be in line with the abutments and piers of the old bridge, and the widths of the piers were similarly governed. There was, therefore, no choice in the fixing of the spans of the new bridge.

The superstructure consists of numerous steel main girders, of the parallel-flanged lattice type, laid longitudinally, with flooring laid transversely on the top flanges (Fig. 3, Plate 1). The main girders and the flooring are entirely below rail-level, or rather below formation-level. Above the flooring the surface is ballasted all over to a minimum depth of about 6 inches below the sleepers of the permanent way.

The superstructure of the old railway-bridge, which is 50 feet wide, is composed of two longitudinal main outside girders, 20 feet in depth, and between these main girders there are cross girders with longitudinal rail-bearers (Figs. 4 and 5, Plate 1).

The new bridge is of such abnormal width that the possibility of its being constructed with a high longitudinal main girder on either side, and cross girders between, was absolutely precluded. In order to allow of the permanent way of the cross connecting lines and junctions of the station-yard being of the Caledonian Railway standard pattern, and being laid in any required position on the new bridge, it was necessary to have an uninterrupted upper surface or flat top on which ballast, together with the other permanent-way materials, could be laid all over; and the design of the superstructure was governed accordingly.

The levels of the undersides of the main girders were also fixed by Act of Parliament. In determining the heights of the girders over the river and the quay on either side, regard had to be had to the requirements of the Trustees of the Clyde Navigation in respect of the navigation of the river and the uninterrupted use of the quays underneath the bridge. The level of the bottom flanges of the girders of the centre span was fixed in relation to the maximum height of the funnels of the river-craft which pass up and down the middle channel, and the levels of the bottom flanges of the girders of the side river-spans were determined by the working headroom required above the quays. The clear heights of the girders of the span over the street on each side of the river were fixed by the City Corporation to be the same as the clear height of the girders of the existing bridge, namely, 20 feet above the surface-level.

The total live load on the bridge may sometimes be very high, because the bridge carries the main part of the station-yard, and the several lines are frequently being run on simultaneously by numerous locomotives and trains. Then, by reason of the weight of the ballast and other permanent-way materials and the limited depth for construction between rail-level and the statutory levels of the bottom flanges of the main girders, with the resulting indifferent proportions of the main girders in respect of the ratio of depth to span, the flanges of the main girders are of necessity large in sectional area and the girders are consequently heavy. Massive piers are therefore required, and the gross weights transmitted to the foundations are very heavy.

FOUNDATIONS.

Geological Investigation of the Site.—It was hoped that the foundations might be constructed on rock, but a series of trial borings revealed the rock to be at such a depth as to make founding on it practically prohibitive. The borings, which were ordinary chisel-borings, were carefully made by a trustworthy boring-contractor, now deceased, and it is pleasing to put on record the accuracy of his work. The character and levels of the geological strata as indicated by the borings were identical with what were revealed in the actual excavations for the foundations (Figs. 2, Plate 1). An engineering assistant was constantly in attendance at the site while the borings were being made, and the results were immediately plotted in section as they were ascertained. Where the character of the strata appeared to change suddenly, confirmatory second borings were always made; such second borings were not infrequent, although they were always found to be corroborative of the first, the change of character being invariably ascertained to be due to a natural fault. The circumstances of the site of the Clyde Bridge rendered trial-pits prohibitive.

Borings, particularly chisel-borings, are proverbially fallacious, but their accuracy in this case was so remarkable as to suggest complete justification of the practice of chisel-boring, as distinguished from boring by the diamond drill, for all the preliminary purposes of engineering work, except the puddle-trenches of waterworks and perhaps the foundations of dock-walls. And the great importance of ensuring accurate results by boring suggests that all qualified boring-contractors should be officially accredited by a recognized authority.

There is some variation in the character of the strata at the sites of the several piers, but at the centre of the river the following may be taken as a typical section:—

	Feet below High-Water Level.
Low-water level	11
Top of mud, river-bed	23
,, muddy sand and clay	25
,, clay and sand	38
,, sand	45
,, fine sand	50
,, coarse sand	58
,, sand and gravel	64
,, sand	71
,, sand and gravel	83
,, rock	92

After careful consideration, the foundations were designed on the assumption that the rock level would not be reached.

Foundations of the Piers.—The construction of the foundations of the piers—land piers as well as river piers—was perhaps the most interesting part of the work. These foundations were constructed by sinking large steel caissons of rectangular form, by the pneumatic process, down to the level of dense sand and gravel.

As an alternative method, it was at first proposed to sink steel cylinders and carry the foundations cylindrically down to the level of the rock, which as already stated, the borings revealed to be about 92 feet below high-water level. Having regard, however, to the comparative difficulty and the inconvenience and uncertainty attaching to working under compressed air with such a head of water, the risk to human life, the possible extra cost, the greater uncertainty as to timeous completion, and the all-important fact that the geological strata at the higher level were quite suitable for founding on, it was ultimately determined to construct the foundations on the sand and gravel, and not on the rock at the lower level. The steel caissons being filled with concrete, the foundation-courses of the several piers are really large monolithic masses of concrete enclosed in skins of steel.

In order to ensure safe loads on the subsoil and have a margin of stability, it was found necessary to make the foundation-courses 4 feet 9 inches wider than the bottom diameter of the cylindrical pillars of the piers, that is 2 feet 4½ inches wider on either side. The extreme width of the caissons for the river piers and land piers had, therefore, to be 23 feet 3 inches and 20 feet 3 inches respectively, measured over the cutting edge. Execution in short lengths being the first principle in the construction of deep and otherwise difficult foundations, it was considered inexpedient to have only one caisson for each pier. A single caisson for each pier of the length which would have been required would have been unwieldy and it might have been difficult to sink it uniformly. A single caisson for the northernmost river pier, for example, would have measured about 132 feet in length by 23 feet 3 inches in width. It was therefore decided to have two caissons for each pier, but, when the work of sinking these was well on towards completion, it was determined to widen the bridge slightly at either end on the east side by "bell-mouthing" in plan, and this necessitated the sinking of two very small additional caissons, one at the east end of each of the land piers. Altogether the caissons were ten in number, and were designated and placed as shown in *Figs.* 7 (p. 16), the two additional small caissons being A3 and D3. The largest two caissons were 89 feet 6 inches long by 20 feet 3 inches wide and 79 feet 4 inches by 23 feet 3 inches.

In designing the foundations regard was had to the necessity of having them at such a depth as to preclude their being affected by scour of the river-bed or by deepening for navigation purposes. A homogeneous mixture of strong sand and gravel, if deep down, is for all the practical purposes of foundations almost as good as solid rock. The mixture of sand and gravel upon which the piers were founded is strong and homogeneous in character. In deciding to found at the higher level on sand and gravel by means of caissons, instead of by sinking cylinders down to the rock, there was due regard to the general economics, and it was kept in view that the sustaining-power of rock might be taken to be much greater than the sustaining-power of a mixture of sand and gravel, and that the areas of the foundations could be proportioned accordingly. The depth had also, however, to be borne in mind; and while, notwithstanding the greater depth, there might have been less volume with cylinders than with caissons, the average unit cost would have been higher; so that in respect of the gross cost it was reckoned that there was little to choose between the two forms of construction. The comparative certainty and safety of the caisson form influenced the decision in its favour, a decision which was proved by after events to be justified, for the caissons were sunk quickly and successfully without untoward incident of any serious kind.

The depths of the bottoms of the main parts of the foundations of the piers, i.e. of the several main caissons, vary from 40·30 feet to 70·47 feet below high-water level. The depths are stated in detail in Appendix I.

The variation in the depths of the caissons of the several piers is due to the different levels of the materials considered suitable for founding on, finally determined as the excavations proceeded, but subject as already stated. The foundations of the land piers were purposely kept much higher than the foundations of the river piers, for, being well protected by the quays, they cannot of course be affected by scour of the river-bed.

In calculating the stability of the foundations, the possible support to be derived from surface friction was entirely disregarded, and with the object of having a greater margin of strength, the part immersion of the piers and the resulting partial diminution of their weight was also to some extent discounted, although the substantially helpful effect of the buoyancy was of course taken into consideration. After careful investigation, it was concluded that dense deposits of sand and gravel at such depths and under the respective circumstances of the piers might be assumed to have safely sustaining-power to the load of about 5 tons per square foot in the case of the land piers and to the load of about 6 tons per square foot in the case of the river piers, and the areas of the foundations were determined on the basis of these unit loads, which however, would be the weights if weighed in air and not in water. The river piers are about 20 to 30 feet deeper than the land piers relatively to the level of low water, hence the different unit load.

The areas of the foundations and the gross and unit loads at the bottoms of the main caissons, disregarding surface friction and buoyancy, are shown by the Table on p. 11.

If allowance were made for the bearing value of frictional resistance at the sides and ends of the caissons, the unit loads in this Table might be stated at about 10 per cent. less; and they would be nearly 20 per cent. less still if the full allowance were also made for the effect of buoyancy. Surface friction is, however, very variable and altogether too indefinite and uncertain to be relied on, particularly in connection with cylinders and caissons of steel, the surfaces of which are smooth; and therefore it should not be a prime factor in the calculation of bearing-areas. Buoyancy is a more tangible thing, and it should always, of course, be given a definite and

Pier.	Bearing-area at Bottom of Caissons.	Gross Load due to dead weight and live load at Bottom of Caissons.	Unit Load due to dead weight and live load at Bottom of Caissons.
1. North land pier—	Square Feet.	Tons.	Tons per Square Foot.
Caisson A 1	1,812	7,481	4·12
Caisson A 2	1,249	5,911	4·73
2. North river pier -			
Caisson B 1	1,845	10,446	5·66
Caisson B 2	1,215	7,247	5·96
3. South river pier—			
Caisson C 1	1,718	10,388	6·04
Caisson C 2	1,129	6,921	6·12
4. South land pier—			
Caisson D 1	1,127	5,355	4·75
Caisson D 2	1,127	5,355	4·75

substantial value in determining the safe loads on submerged foundations. On the other hand, in connection with deep foundations not in water it is somewhat unusual for the initial pressure of the original superincumbent earth to receive much consideration. Neither is it very customary to make any allowance in respect of water displaced in deep foundations in strata which are water-bearing—permeable strata such as running sand, for example—although in such a case the structure may be said to be to a certain extent water-borne, there being no uncertainty as to the helpful effect of the diminution of the weight of that part of it which is below the level of the surface of the subsoil-water. With a pier or other structure of constant cross-sectional area below water-level, this helpful effect is of course identical with the intensity of the hydrostatic pressure at the base due to the head of water.

The conditions and circumstances attaching to deep foundations are so very varied and difficult as perhaps to make it almost idle to sift and tabulate recorded experience with the object of standardizing the principles and practice in respect of the calculation of safe loads. There appears, however, to be such a want of uniformity that something might be done in this direction with very great advantage.

Caissons.—The caissons were of wrought steel and, although they varied in size, the scantlings of the plates and bars were the same in them all. They are typically shown in Figs. 8 and 9, Plate 1, inclusive. The working-chamber was 8 feet high, and access to it was got by means of shafts 3 feet 6 inches in diameter. There were three access-shafts in the larger main caissons and two in the smaller, and they were so situated as to be in the centre of the cylindrical masonry pillars. There was only one access-shaft in the subsidiary caissons A 3 and D3.

The shell-plates of the caissons ranged in depth from 3 feet 11½ inches at the top of the caisson to 4 feet 11 inches at the roof of the working-chamber, but they were 8 feet 4 inches deep in the sides and ends of the working-chamber. They ranged in thickness from ⅜ inch at the top to ½ inch at the bottom, and the plating of the working-chamber was also ½ inch thick. The shell-plates overlapped at the horizontal joints and receded towards the top, and were well stiffened by longitudinal angles 5 inches by 3 inches by ½ inch. The vertical joints of the shell-plates and of the plating of the working-chamber were butt joints with ⅜ inch covers. The caissons were also stiffened by horizontal diagonal bracings in the form of angles 4 inches by 4 inches by ½ inch, which were subsequently built into the masonry, and further stiffening towards the bottom was afforded by horizontal girders placed transversely at the roof of the working-chamber, about 5 feet apart, between centres. The primary purpose of these girders, however, was to provide support for the ceiling-plates of the working-chamber, which were also the floor-plates of the caisson and, as such, carried the concrete filling of the caisson pending the deposition and packing of the concrete in the working-chamber. Underneath these horizontal girders there were triangular stiffening-diaphragms, which extended down to and stiffened the cutting edge.

The cutting edge was a plate, 15 inches deep and 1 inch thick, placed on the outside and riveted to the bottom shell-plate through two internal continuous stiffening-angles 4 inches by 4 inches by ⅝ inch. The bottom of the plating of the working-chamber was curved round and riveted between these continuous stiffening-angles, and the angles, together with the plating and its curved and blunt form at the bottom, offered desirable retarding influence in the sinking and tended to prevent any sudden drop. The cutting edge projected its full thickness externally, and this, with the shell-plates receding from bottom to top, gave the sides of the caissons a batter of about 1 in 100, the effect of which was to somewhat lessen the surface friction and facilitate the sinking.

The rivets of the caissons were ¾ inch in diameter and 3 inches pitch, excepting in the access-shafts and working-chamber, where, having regard to the effect of the high-pressure air, they were ¾ inch in diameter and 2¼ inches pitch, and in the cutting edge, where they were 1 inch in diameter and 4 inches pitch. The outside rivet-heads were hammered nearly flat, except on the outside of the cutting edge, where they were countersunk so as to lessen the friction in sinking. In order to ensure air-tightness, the joints of the working-chamber and the access-shafts were thoroughly caulked by hammering the edge of the plate into the surface against which it rested and the caissons were made watertight in a similar manner.

The over-all dimensions and weights of the steelwork of each of the main caissons of the several piers are stated in Appendix II.

The heights of the caissons, and to some extent the lengths, were determined in relation to the character of the strata suitable for founding on, but the exact lengths were governed mainly by the positions of the shafts of the working-chamber, which in their turn were fixed by the positions of the pillars of the piers.

THE MASONRY OF THE SUBSTRUCTURE.

The abutments of the bridge are ordinary brick walls built on a concrete foundation-course, and, in accordance with statutory requirement, they are faced towards the streets with bricks of white enamelled surface. They are pierced by arched openings which give access to premises situated underneath the train-shed of the station and the station-yard, on the north and the south side of the river respectively.

There was no statutory obligation as to the number of pillars in the land piers, but there was restriction in respect of the river piers, and, in conformity with the Act of Parliament, there were five pillars in each of the river piers. There were five main pillars in the north land pier and four in the south land pier, but the "bell-mouthing" of the bridge on the east side at each end necessitated the construction of a subsidiary pillar and some additional masonry in each of the land piers. The pillars of the north and south river piers are 27 feet 1½ inch apart between centres and 25 feet 3½ inches between centres respectively, and the main pillars of the north and south land piers are 31 feet 8 inches apart between centres and 29 feet 7 inches apart between centres respectively. The lengths of the north and south river piers, measured just below the bed of the river, are about 127 feet and 120 feet respectively, while the lengths of the north and south land piers, measured just below the level of the quays, are about 158 feet and 117 feet respectively. The pillars are constructed of composite masonry in cement mortar, built and well distributed by footing-courses on the concrete monoliths. The internal masonry of the main part of the pillars is red brickwork with several through strengthening-courses of blue brickwork, and there is some concrete hearting in the form of filling of the access-shafts of the caissons. The character and form of the composite masonry of the pillars is shown by Figs. 2 to 6, Plate 1.

The pillars are faced with blue brickwork up to within 6 feet of low-water level in the case of the river piers and to within 2 feet of the surface of the quays in the case of the land piers. Above these levels the facing is of light grey granite ashlar of rock-face dressing up to high-water level and fine-axe dressing above. At the level of the top of the blue-brick facing these diameters are lessened by 1 foot 6 inches. As the pillars rise, the diameters become further reduced, for, having regard to appearance, they are built with a batter. In order to secure further architectural effect, the pillars of the river piers are connected above high-water level by granite arches of pleasing form, and there are also connecting arches between the pillars of the land piers. These latter, however, are more utilitarian in their purpose, being of brickwork and built below the level of the surface of the quays for the purpose of acting as a foundation for brick screen-walls between and in front of the pillars of the land piers. These screen-walls are also faced with bricks of white enamelled surface, under statutory obligation.

The concrete, which was machine-made, was mixed in the proportion of one part of cement to five parts of ballast, the ballast consisting of a mixture of equal volumes of broken clinker bricks and whinstone or of broken freestone and whinstone with the necessary volume of sand. The clinker bricks and the freestone and whinstone were of hard quality and were broken to an angular shape and of such size as to pass through a screen having meshes 1¾ inch square. The sand was sharp and coarse and was partly river-sand and partly pit-sand, as well as to some extent crushed freestone, the last however, as being the best, being used chiefly in the making of mortar. The main requirements were that the ballast should be thoroughly clean, that the several materials should be properly proportioned, that the aggregate and matrix should be specially well mixed, and that by means of buckets with opening bottoms the concrete should be carefully deposited in layers and be thoroughly rammed in situ with the object of ensuring density.

In the brickwork the red bricks were clay bricks as distinguished from composition bricks and were of a size to build four to the rising foot. They were built in English garden bond, and a dense mass was ensured by laying and "floating" the bricks in a deep layer of mortar which rendered grouting unnecessary and gave that best of all engineering masonry, good brickwork in cement.

Every granite stone used was dressed at the quarry and brought to the site of the bridge ready for building.

The mortar of all the kinds of masonry was cement mortar in the proportion of one part of cement to two parts of sand, and the cement grout was of similar quality, only differing from the mortar in the volume of water used in the mixing. By special requirement the mortar was mixed by hand-labour. The requirements of the specification in respect of cement were very similar to those of the British Standard specification.

In the design of the works of the bridge, as well as of the other works of the station, the unit loads which it was considered expedient to have on the several kinds of masonry were assumed to be the following:—

	Tons per Square Foot.
Concrete in foundations	7
,, in walls	8
Red brickwork in cement	10
Blue ,, ,, ,,	15
Freestone square rubble masonry in cement	8
,, ashlar masonry in cement	15
Granite ,, ,, ,, ,,	25
Freestone ashlar bearing-blocks	12
Granolithic bearing-blocks	12
Granite ashlar ,,	20

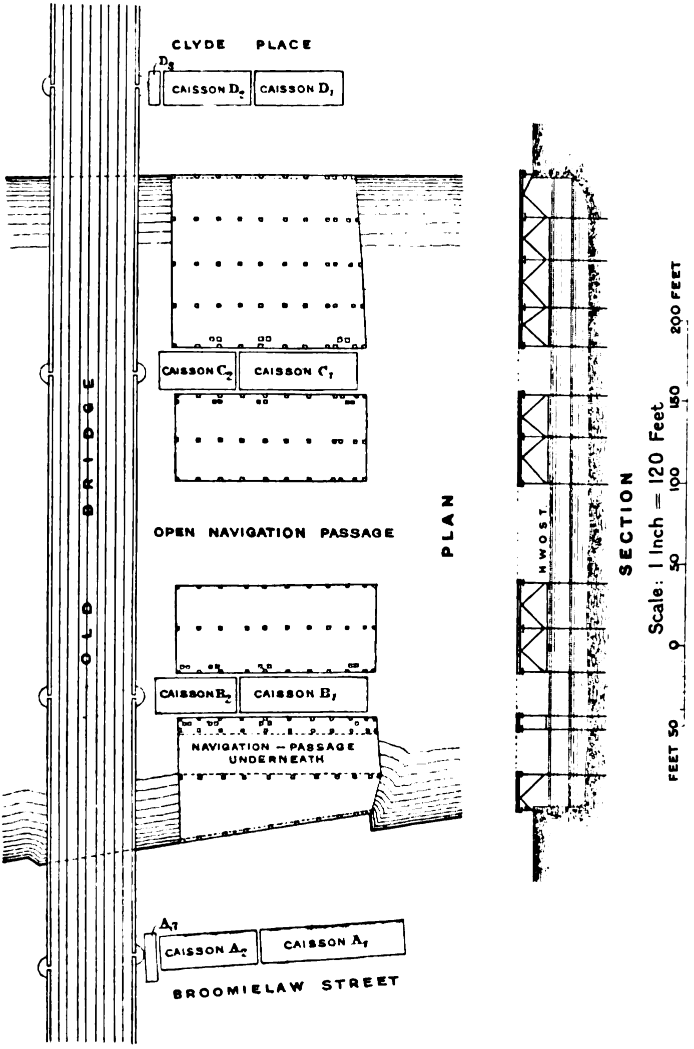
These are moderate loads giving considerable margin of stability, and while it cannot be said that they were strictly adopted, they served as standards. The weights of the several kinds of masonry were ascertained by careful experiment to be as stated in Appendix III.

The Temporary Staging.—The Company were authorized by the Act to erect, across the river and quays, a temporary staging of the full width of the new bridge and 21 feet wider on the west side. The only important requirement of the Clyde Navigation Trustees was that there should be two open passages underneath the staging for navigation, one on the north side adjoining Broomielaw Quay and the other in the centre of the river. The former was made 30 feet wide, while the latter was made 62 feet wide. Both passages were well lighted at night by electric lamps. The staging is shown in skeleton plan and longitudinal section by *Figs. 7* (p. 16).

The headroom of the open navigation-passages had to be the same as the headroom of the arches of Glasgow Bridge, that is, about 19 feet above high-water level.

The maximum width of the staging was about 125 feet, and its main purpose was to provide such platform accommodation over the river and quays as would facilitate erection and sinking of the caissons, erection of the main girders, and construction of the other works. The piers of the staging were placed in such positions as to allow of the caissons being sunk and the other works of the

Figs. 7.


CLYDE PLACE
D3
CAISSON D2
CAISSON D1
CAISSON C2
CAISSON C1
OLD BRIDGE
OPEN NAVIGATION PASSAGE
PLAN
CAISSON B2
CAISSON B1
NAVIGATION — PASSAGE UNDERNEATH
A3
CAISSON A2
CAISSON A1
BROOMIELAW STREET
H W O S T
SECTION
Scale: 1 Inch = 120 Feet
FEET 50
0
50
100
150
200 FEET

Temporary Staging.

permanent bridge conveniently executed without modification of the staging, and the main uprights of the piers of the staging were placed on the lines of the main girders of the permanent bridge, so as to be directly under the heaviest loads which the staging was meant to carry.

The piers were in the form of a single row of pitch-pine timber piles, except those adjoining the open navigation-passages, which were formed of a double row of piles. The piles, which were 12 inches square, were driven to an average depth of 12 feet below the bed of the river, and they were thoroughly braced from about mean-tide level upwards. Except at the navigation-passages, the spans of the staging were about 27 feet, and transversely the piles were spaced about 12 feet apart between centres in the single as well as in the double rows.

The flooring of the staging, which was of 3-inch planking laid transversely, was carried on timber beams laid longitudinally from pier to pier of the staging directly on the line of the piles, while in the case of the central navigation-passage the flooring was carried on steel girders. These girders were only six in number, as with a view to economy only part of the central passage was covered at one time. At places on the staging where certain structures had to be erected for the temporary support of the caissons of the river piers during their erection and for the purpose of pitching the caissons, the main staging was strengthened by driving extra piles and laying down additional longitudinal beams as described later on. The measure of success in bridge-construction is governed by the character of the temporary staging.

THE PNEUMATIC INSTALLATION.

Having regard to the character of the foundations of the piers of the bridge and the desirability of constructing them by working under compressed air, the importance of having an efficient pneumatic installation was early and fully recognized.

The contract obligations, in respect of the pneumatic installation, as they were stated in the Specification, are given in Appendix IV: but, on account of the moderate air-pressure, the requirements were not arbitrarily insisted on.

The principal air-compressing plant was situated at Clyde Place on the south side of the river. It consisted mainly of a boiler, an air-compressor and an air-receiver. The steam- and air-cylinders of the compressor were 16 inches and 14 inches in diameter respectively, with a stroke of 30 inches, and the air-receiver was 6 feet 6 inches in diameter and 30 feet long. There was also an auxiliary air-compressor, for use in emergency, situated on the north side of the river, but the main air-compressing plant proved very satisfactory, and the auxiliary compressor was required on only one occasion.

The air-piping between the main compressor and the receiver consisted of cast-iron flanged pipes 4 inches in diameter controlled by 4-inch valves. The outlet-piping from the air-receiver was 3 inches in diameter and also of cast iron with flanged joints. The piping was carried on the temporary staging and was led to each pier and laid longitudinally in front of each caisson. The loss of air in transmission from the receiver on the south side of the river to the caissons on the north side of the river was about 5 lbs. per square inch, presumably due to friction and leakage. In order to protect it from frost, the piping was placed in wooden casing of rectangular section and packed with stable-litter. The pipes were, however, one day frozen to such an extent that there was a stoppage of several hours.

The pneumatic plant further included an installation of air-locks, three for the passage of workmen, and three for the passage of materials, and these were attached to the access-shafts of the caissons. The locks were constructed of wrought steel. The material-locks attached to each caisson were always in use, but usually only one man-lock was used in connection with each caisson, the other being only provided for use in case of emergency. The locks were attached to the tops of the access-shafts.

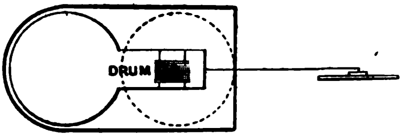
The character and relationship of the man-lock and the material-lock, with the attachment to the access-shaft of the caisson, are shown diagrammatically by *Figs. 10.*

The two locks formed one steel structure. Passage between the outer and inner chambers of the locks was by way of steel doors, all of which had beadings of indiarubber closing against the plating of the sides of the chambers, to ensure air-tightness. The man-lock was bolted tightly to the top of the access-shaft of the caisson, for which purpose the short length of shafting, V, was fixed permanently to the lock.

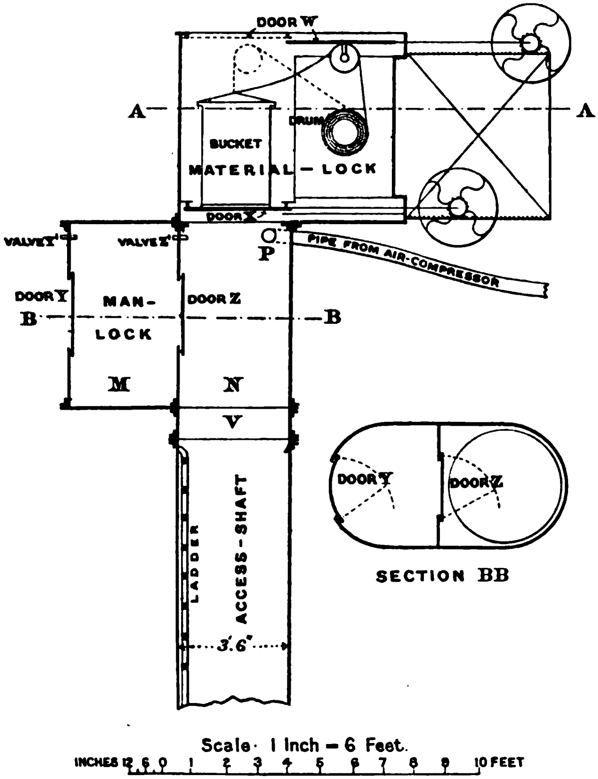
The inner chamber, N, of the man-lock, which was circular and 3 feet 6 inches in diameter, really formed part of the access-shaft, but for convenience in description it is designated as the "inner chamber." Compressed air was led from the air-receiver to the top of the inner chamber and access-shaft of the caisson by the pipe P, and valves (Z′ and Y′) of such small capacity as would only allow of slow passage of air were placed between the inner and outer chambers and between the outer chamber and the atmosphere. These valves regulated the air-pressure in the opening and shutting of the doors of the chambers.

In order to leave the outside door Y open after passing through the lock, the last man of an ingoing gang, before leaving the outer chamber, would open the valve Y′ and close the valve Z′ and then, getting quickly on to the ladder in the inner chamber, would

Figs. 10.



SECTION AA SHOWING DOOR X OPEN


DOOR W
A
A
BUCKET
DRUM
MATERIAL — LOCK
DOOR X
VALVE Y′
VALVE Z′
O
P
PIPE FROM AIR-COMPRESSOR
DOOR Y
MAN-LOCK
DOOR Z
B
B
M
N
V
LADDER
ACCESS-SHAFT
3′.6″
DOOR Y
DOOR Z
INCHES 12 6 0 1 2 3 4 5 6 7 8 9 10 FEET

SECTION BB

Scale · 1 Inch = 6 Feet.

AIR-LOCKS.

immediately close the door Z. Before one could get out of the access-shaft and inner chamber, someone from the outside had to enter the outer chamber and equalize the pressure in the outer chamber with the pressure in the inner chamber by shutting the door Y and the valve Y′ and opening the valve Z′.

The material-lock was also in the form of an outer chamber attached to the inner chamber, but, whereas access from the man-lock to the inner chamber was by the door Z at the side, access from the material-lock was by the door X in the top.

Circular buckets were used for raising the excavations from the working-chamber (*Figs. 10* and *12*). In lowering the empty bucket, it was first deposited in the open outer material-chamber, by means of a crane, on to the top of the closed door X between the outer and inner chambers. The crane-chain was then unhooked from the bucket and the rope from a pulley which hung on the outer door W, and which was attached to a revolving drum inside the lock, was hooked to the bucket. The outer door W was now moved into position for shutting by means of a horizontal rack and pinion worked by an outside wheel, as shown by the dotted line on the diagram, and was then by means of an outside lever lifted into position with its rubbers hard against the flange. The compressed air was thereafter allowed to rush from the inner to the outer chamber by means of a large valve operated from the outside, and the air-pressure in the inner and outer chambers being thus equalized, the door X was lowered by an outside lever from its bearing on the flange and moved out by means of a horizontal rack and pinion worked by an outside wheel, as shown in *Figs. 10.* The drum in the material-lock was operated by a steam-engine attached to the outside of the lock, and communication between the outer and the inner chamber and access-shaft having been effected as described, the man in charge lowered the bucket to the working-chamber by unwinding the drum. When the bucket was filled and ready to be raised the operation was reversed.

Each of the working-chambers of all the caissons was lighted by means of three 32-candle-power electric glow-lamps. The wires were led through stuffing-glands into the outer and inner chambers of the air-lock and were taken down the access-shafts at the back of the ladder.

Signalling between the working-chamber and the outside of the locks was effected by means of an outside whistle with rope attached, the rope hanging down the full depth of the access-shaft. There was a code of signals for the several operations.

Except in connection with caisson C 1, which is founded at a depth of 70 feet below high water, the air-pressure was of moderate intensity. There was therefore little necessity for special concern in respect of the care of the workmen as, although they worked in shifts of 12 hours, divided into three separate periods by two meal-hours intervening, they enjoyed good health and liked to be employed below. The 10 hours' work was made up of two periods of 3 hours each and one period of 4 hours, the operations being carried on by relays of men working night and day.

In the last stages of the sinking of caisson C 1, with an air-pressure of 32 lbs. per square inch, there were some complaints of pains in the joints owing, it was thought, to too rapid passage through the air-locks. The Author's experience is that, in spite of all remonstrance and warning, workmen when passing through the air-locks, frequently equalize the pressure by means of the "muck-tap." The worst effects of the compressed air were observed when the caissons were in the clay, and this notwithstanding that the maximum pressure was not at the time higher than about 22 lbs. per square inch. With clay in the bottom, free passage of the air underneath the cutting edge was prevented, so that the atmosphere of the working-chamber became bad and exercised a fatiguing influence on the men, which, coupled with the harder excavating work, tended to exhaust them on the long shift. With the cutting edge in sand and gravel, however, the air could escape, so that in some degree there was then continual change of air. The men were also troubled somewhat during the concreting of the working-chamber, for as the concrete was deposited the place became small, the temperature high, and the air vitiated, the effect of which, and of their cramped position in working when nearing the roof, was to cause them much discomfort, though it cannot be said that there was actual suffering.

Work under compressed air was commenced in October, 1901, and was carried on until April, 1903. Some of the men worked under compressed air throughout the entire 18 months and appeared to be none the worse.

The locks, caissons, engine-room and offices were lit by electric light, the current being obtained from the Glasgow Corporation public supply, but, having regard to the inconvenience there would have been in swinging cranes in the vicinity of electric overhead wires, the outside works were lit mainly by acetylene gas.

CONSTRUCTION OF THE PIERS.

In the process of sinking the caissons the external weight necessary to overcome the surface friction of the sides, the upward force of the high-pressure air in the working-chamber, and under certain circumstances perhaps some resistance of the cutting edge was not provided by means of special kentledge, but the caissons were at once loaded with the permanent masonry and sunk by the weight which it afforded. The masonry was built on the caissons as additional weight was needed, and as lowering proceeded, the work of excavating in the working-chamber being of course carried on simultaneously.

There being piers on the land and piers in the river the circumstances of the respective caissons were different, so that in the details the sinking operations varied. The main caissons for all the piers were built up and riveted complete in situ directly over their permanent sites, those for the land piers in excavated trenches, and those for the river piers on the temporary staging.

The Land Piers.—Trenches with sloping sides were excavated about 8 feet deep, measuring from the surface of the quays. These trenches were at the bottom 4 feet longer and 4 feet wider than the length and width of the caisson, to ensure convenience in the erecting and riveting of its steelwork. Resting on timbers arranged in the bottom of the trench, the cutting edge of the caisson was about 2 feet above the bottom of the trench, and this enabled the workmen readily to obtain access in the riveting and caulking of the working-chamber. The plates and bars were placed in position by means of two hand-cranes on the surface, the jibs of which were of such radii as to sweep the entire area of the caisson and trench. The caisson was thus gradually built up, riveted, and caulked, and, when completed to a height of about 16 feet above the cutting edge, it was raised from the timbers and carefully lowered into its exact position at the bottom of the trench by means of hydraulic jacks. Some lengths of access-shafting were then fixed, and the caisson was ready for sinking.

The four main caissons of the land piers (A 1, A 2, D 1, and D 2, *Figs.* 7) were erected and pitched in this way. The smaller caissons (A 3 and D 3) were similarly pitched, but in erecting them the working-chambers were not built in situ: being comparatively small, and only weighing about 20 tons, they were built up and riveted in the bridge-yard, and so brought to the site.

In the sinking of the several caissons of the land piers the operations were practically identical. Compressed air was not used in the first stages, for, although the caissons were pitched in trenches about 8 feet deep, the cutting edge was still about 9 feet above the level of the subsoil-water, so that to the extent of this latter depth the excavation in the working-chamber could be continued in the dry. It is remarkable that, although the caissons of the land piers were sunk at such short distances as 65 feet and

45 feet from the edge of the quay on the north and the south side of the river respectively, the level of the subsoil-water was never found to be higher than about 7 feet below high-water level, that is 4 feet above low-water level, and it did not appreciably vary with the level of the tide.

Immediately following the pitching of a caisson, concrete was deposited in it between the diaphragms at the sides and ends of the working-chamber and ultimately over the working-chamber and upward, as the steelwork of the upper part of the caisson was erected, and as sinking proceeded. The concrete was mixed mechanically, and was deposited by means of iron buckets of 1 cubic yard capacity, lifted and lowered by a steam-crane. The buckets had opening bottoms, and the concrete was thereby handled with a minimum drop, laid in layers, and thoroughly rammed. There was some inconvenience attaching to the deposition of the concrete in the caissons, as the buckets had to be carefully lowered between the bracing-bars.

Simultaneously with deposition of the concrete in the caisson, excavating operations were carried on in the working-chamber, the excavated materials being removed from the chamber through the open access-shafts by means of the circular buckets lowered and raised by steam-cranes. Care was taken to ensure that the caisson should be kept sinking while the concrete was being deposited, as otherwise, with the centre of gravity of the loaded caisson gradually rising, there would have been risk of the caisson canting, so that relatively it was kept deeply embedded in the ground.

The caissons were erected to their full height, and the concrete was entirely deposited from above the working-chamber upward, before sinking by the pneumatic process began, as, with the air-locks in operation, and having regard to convenient use of the cranes, it would have been impracticable to have been depositing concrete in the caisson simultaneously with excavating in the working-chamber.

The main caissons ranged in height from about 41 feet to about 52 feet, their actual height depending on the depth from a level just above subsoil-water level down to the level of the strata suitable for founding on. At the stage of the operations, therefore, when the cutting edge was just at subsoil-water level, the bottoms of the several caissons were about 18 feet below quay-level, so that the average level of the tops of the caissons, when sinking by compressed air began, was about 28 feet above quay-level.

With the cutting edge at subsoil-water level, the locks and other pneumatic appliances were installed, and the further sinking of the caissons was carried on under compressed air. In the sinking of the caissons of the land piers, the cutting edge was usually 12 inches to 18 inches deeper than the level of the excavations in the working-chamber, although at times the excavations in the working-chamber were taken out quite as low as the cutting edge, as the surface frictional resistance being considerable, there appeared to be no risk of sudden drop. If anything, the cutting edge was relatively lower when in the clay, but the clay was found more difficult to excavate than the sand and gravel, because the action of the compressed air on the clay was such as to make it quite stiff and hard on the surface, and this to such extent, indeed, that special spades had to be used in its removal instead of the ordinary excavating shovel. The frictional resistance was found to be greater in the sand and gravel than in the clay, as the clay was of a muddy and sandy character, and was naturally soft.

As already mentioned, the deposition of the concrete in the caissons was completed before sinking by the pneumatic process was commenced. In the case of caisson D 1 of the southernmost land pier, the concrete was finished to a height of about 12 feet above the top of the working-chambers of the caissons, that is, to about 20 feet above the cutting edge. This, with the caisson and the air-locks represented a total weight of over 1,000 tons, but was not found sufficient to carry the caisson farther down than to 20 feet below the bottom of the trench in which it had been pitched, out of a total depth of 48 feet below the bottom of the trench to which it had to be sunk. The necessity for greater weight was met by building the brickwork of the pillars of the piers, and by depositing the excavated material from the working-chamber as high as possible in the caisson on the concrete and around the brick pillars. In the later stages of the sinking, however, even with such additional weight, there was sometimes difficulty in forcing the caissons down. This was particularly the case when gravel was reached, and in such ground they became stubborn in the extreme, and refused to move. In these circumstances it was the practice to thoroughly clear out the working-chamber down to the cutting edge, and thereby lessen its bearing all along the sides and ends, as the inclined part of the roof of the working-chamber and the blunt edge of the bottom offered considerable but salutary resistance to sinking. This operation assisted the downward movement, but to accelerate it the high-pressure air in the working-chamber which, of course, tended to keep the caisson up, was at times temporarily reduced during meal-hours, when the men were out, by allowing it to blow off through the large valve in the material-lock after cutting off the supply, the effect of which was to assist the caisson to sink. After an operation of this kind, when access had again been got to the working-chamber of a caisson, the sinking of which had been temporarily suspended, it was found that the earth in the working-chamber was roof-high, the material having been brought in by water from beyond the sides of the working-chamber underneath the cutting edge. To this circumstance was attributed certain subsidence of parts of the surface of the quays in the vicinity of some of the caissons of the land piers.

The practice of reducing the air-pressure in the working-chamber was also adopted to right the caissons when they had got slightly off the level. Material was heaped and pounded against the lower side or end of the working-chamber, and sometimes pieces of wood were inserted below the cutting edge, which, with the reduction of the air-pressure, had the effect of lowering the higher side or end.

Immediately the sinking of the caissons became difficult, the air-locks were transferred temporarily to another caisson while the brick pillars were being built on the caisson requiring additional weight. Simultaneously with the building of the brickwork, excavations from another caisson were being heaped round the pillars to give more weight, and lengths were being added to the access-shafts in order to keep the mouths of the shafts always above the masonry. In other words, the operations were so arranged as to ensure that all the plant should be as constantly in use as possible. In the case of one of the caissons of the south land pier only one stop was made in the sinking under compressed air, and the brickwork was built during that stop to within a few feet of the height at which it was calculated the granite facing should begin after the caisson was sunk. In other caissons the brickwork of the pillars, and even of the arches between the pillars, was entirely completed before the last stage of the sinking of the caissons was commenced.

Having regard to the experience gained in connection with the caissons of the south land pier, it was determined to increase the quantity of the concrete in the caissons on the north land pier and thereby add to the weight for the purpose of facilitating the sinking. Concrete was therefore deposited 5 feet higher, that is, to 25 feet above the cutting edge. During the sinking of caisson A 1 through a thick bed of gravel, one end of the caisson got into sand, and as there was indication of the caisson tending to settle down at the sandy end, the sinking was subsequently stopped, and this caisson was founded at a level about 11 inches above the proposed foundation-level shown on the contract-drawing.

Notwithstanding the great care exercised in the sinking of the caissons of the land piers and the daily checking by levelling and otherwise, there was found to be some deviation from absolutely true line and level when the sinking was finished. The deviation was, however, very slight, never being more than 1 or 2 inches in level and 2 or 3 inches in line, and it was of little consequence, for no part of the granite work of the land piers was commenced until both the main caissons of each land pier had been sunk and the brick pillars built. There was therefore time and space for adjustment, as the brick pillars had ample projecting scarcement.

When the caissons had been sunk to the required levels, concrete was deposited in the working-chambers and access-shafts by means of the circular buckets, having opening bottoms. Experience of the difficulty of sinking the caissons in the sand and gravel having suggested that there was little risk of sudden drop and further downward movement, it became the practice in the case of the land piers to remove the excavations entirely and to clean out the working-chambers before any concrete was put in. The method of depositing the concrete in the working-chambers was in all cases the same. It was first deposited on a timber platform laid immediately below the access-shafts, and then, beginning at each end of the caissons, it was gradually spread in layers over the several areas between the access-shafts and the ends, until the working-chamber was filled to within about 12 inches of the roof. For a time it was kept as low as possible directly underneath the access-shafts, to give room for the men to work; but these places were also filled gradually, the men working themselves up into the shafts. Latterly, the men only went down to spread and ram the concrete as each third bucketful was deposited.

The packing of the concrete hard up to the roof of the working-chamber was considered to be of vital importance, and was carefully carried out in short sections and in very thin layers, each being allowed to set partially before the next was deposited. As the roof of the working-chamber was approached, grout was added, in order that the concrete should be made perfectly solid. Depressions about 6 feet in diameter and 1 foot deep were latterly formed in the concrete, directly underneath the access-shafts, into which grout was poured until it stood to a height of about 3 feet in the access-shafts. Latterly the concrete was dumped down into this liquid grout, and the filling of the access-shafts being done in a similar manner, the air-pressure was gradually reduced and ultimately withdrawn.

The River Piers.—The caissons of the two river piers were erected on the temporary staging as nearly as possible over their permanent positions. Particular care had to be taken in the setting out and sinking of the caissons, for it was intended to build the whole of the brickwork and part of the granite work of each pillar before the sinking was completed. There are two caissons in each pier, on one of which there are three pillars, and, on the other, two. With this distribution of the pillars, the necessity for accuracy in alignment will be appreciated, particularly when it is borne in mind that there were nineteen pillars in the land and river piers, several of which were being built and sunk simultaneously.

The temporary staging had been constructed so as to allow of the caissons being built on it and thereafter lowered between its piers; that is, there were openings in the staging to allow of the caissons being lowered into their permanent positions. Extra piles were driven to strengthen the staging in the vicinity of the openings, and additional beams, 12 inches square, were laid over each opening, on which 9-inch by 3-inch planks were placed transversely at the level of the staging flooring, and on these timbers the lower part of each caisson was erected.

It was essential that the erection of each of the caissons to its full height, as well as the caulking, should be entirely completed before the caisson was floated. Special appliances were therefore necessary to raise the caisson from the supporting timbers and thereafter to lower it into the river. With the caisson resting directly on the supporting timbers, it could not of course be lowered into the river without these timbers being removed, so that at some time or other it had to be supported by being hung from overhead. Having regard to the method proposed to be adopted for the suspending, lifting, and lowering of the caisson, it was thought expedient to erect only the lower part with direct bearing on the supporting timbers underneath, and after such lower part had been attached to an appliance overhead and the supporting timbers underneath had been removed, to complete the erection of the caisson as it hung in mid air. The caissons were suspended, lifted and lowered by means of special hydraulic jacks and other appliances.

The main appliances for suspending, lifting and lowering a caisson consisted of four timber trestles, four steel girders, four hydraulic jacks and four steel suspending-bars. Two of the trestles were placed on each side of the opening in the staging, and pairs of steel girders, spanning the opening, rested on the trestles. The hydraulic jacks were supported by the steel girders and were connected with the suspending-bars which hung between the girders and were attached below to certain of the cross girders of the caisson at the roof of the working-chamber (Figs. 9 and 9a, Plate 1).

The trestles were built of pitch-pine timbers 12 inches square. The uprights of each trestle, which were two in number, rested on longitudinal timber sills. The timber sills were supported on the temporary staging, which was strengthened by driving additional piles deep in the river-bed directly underneath.

Each of the two pairs of trestles was placed at about one quarter of the length of the caisson from either end of it. The caisson being 23 feet 3 inches wide, and the opening in the staging 27 feet wide, the width between the trestles was made 28 feet, and this space was spanned by the pairs of steel girders. These girders were 3 feet 6 inches apart between centres, their positions being so fixed as to allow of the suspending bars hanging freely between them.

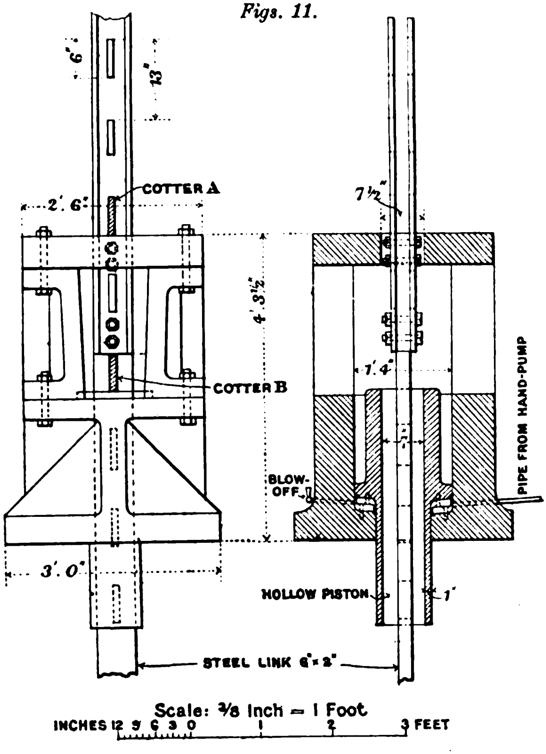
The height of the trestles was about 14 feet above the floor of the staging. The hydraulic jacks were placed over the line of either side of the caisson, resting on pairs of transverse timber beams. Four hydraulic jacks were used to each caisson, two on each of the pairs of overhead girders. The jacks had hollow pistons through which the suspending-bars for hanging the caisson were passed and attached to two of the intermediate cross girders at the top of the working-chamber (*Figs. 11*). The suspending-bars were steel-bar links, each 7 feet 4 inches long and 6 inches by 2 inches in cross section. The links were single-bar and double-bar alternately, the single-bar link being bolted between the double-bar links at either end by four bolts 1½ inch in diameter in double shear. In the links there were slots as shown by *Figs. 11*.

The attachment of the suspending-bars to the caisson was made by means of brackets consisting of steel gusset-plates and angle-bars riveted to the top flanges of the cross girders at the top of the working-chamber, and, having regard to the transference of the heavy weight of the caisson to the suspending-bars, the rivet-area of the attaching bracket was increased by continuing the end angles of the particular cross girders upwards about a foot higher than the normal, in order to permit of horizontal riveting to the end angles as well as vertical riveting to the flange (Figs. 8 and 9, Plate 1).

The lower part of the caisson, to the top of the first plate above the working-chamber, having been erected on the staging and attached to the suspending-apparatus, it was lifted by the jacks to about 2 feet above the timber floor, to allow of the riveting and caulking of the working-chamber being completed.

At the commencement of the lifting-operations, the caisson was hanging on the cotters A (*Figs. 11*). The lifting of the caisson was performed by inserting the cotter B in the first slot of the suspending-bar above the piston of the jack, and, by means of hand-pumps attached to each jack, raising the piston of the jack against the cotter B and continuing to raise it until the slot above cotter B had cleared the upper collar of the jack, when cotter A was inserted, and the weight of the caisson was transferred again to the upper collar

Figs. 11.


6"
13"
COTTER A
2'. 6"
4'. 8½"
COTTER B
3'. 0"
7½"
1'. 4"
BLOW-OFF
PIPE FROM HAND-PUMP
HOLLOW PISTON
1"
STEEL LINK 6"×2"
Scale: 3/8 Inch = 1 Foot
INCHES 12 9 6 3 0 1 2 3 FEET

HYDRAULIC JACK.

of the jack by releasing the piston. By repeating these movements the caisson was gradually raised, and the slots in the suspending-bar being 13 inches apart, each operation lifted the caisson 13 inches.

After the riveting and caulking of the lower part of the suspended caisson had been completed, the timber beams and planking on which it had been erected were removed. While thus suspended, it was built up plate by plate, riveted, caulked and completed, and afterwards gradually lowered between the piers of the temporary staging.

The operation of lowering the caisson was exactly the reverse of that gone through in lifting it. It was very essential that the caisson should always be kept quite level, in order to ensure equal distribution of the load on the four points of support, i.e., the jacks, so that in the lowering as well as in the raising great care had to be taken to ensure uniformity in the simultaneous movements of the four jacks.

To provide against the possibility of flooding, the caissons of the river piers required to be about 39 feet high. Caisson C 1 was the first caisson to be built and sunk, and the experience which it provided suggested that the top tier of plates of the other caissons might be bolted in their attachment instead of being riveted. With the bolted attachment and the caisson getting into material suitable for founding on and not requiring to be sunk farther, the top tier of plates could be taken off if the caisson were not deep enough to meet the requirement of the Clyde Navigation Trustees in respect of depth of clearance. In the course of the sinking of the first river pier, caisson C 1, the clearance was fixed by the Navigation Trustees at 20 feet below low-water level. This bolted attachment was adopted for the remaining caissons, so that while caissons B 1, B 2, and C 2 were about 39 feet high in the first instance, the top tier of plates in each was only temporary and was removed when the masonry had been built to such level, that in the course of the further sinking the building-operations could not be detrimentally affected by flooding. The caissons B 1, B 2, and C 2 are therefore about or rather less than 35 feet in height, but caisson C 1 is about 39 feet in height, and it was sunk to the full depth necessary to give the required clearance below the low-water level. It was ascertained that even with the concrete fully deposited none of the caissons sank in the first instance to a level lower than that represented by the roof of the working-chamber, being about 2 feet above the bed of the river; so that in the case of caisson C 1, there was perhaps excess of caution. The sinking of caisson C 1, it may be said, however, was to some extent experimental, and, as failure might have been disastrous, caution with the first caisson was particularly essential.

While the caissons were still hanging on the hydraulic jacks, a careful survey of the river-bed was made at the site of each pier, with the result that in the case of the north pier the bed was found to be very irregular in level. It being essential that the cutting edge of each caisson should touch the bottom on level ground, the inequalities were levelled by depositing excavated material from one of the other caissons, and uniformity of the river-bed was thus secured.

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Just before the caissons were lowered into the water, five stiffening settings of timber were fixed inside each temporarily to preclude the possibility of the sides of the caisson tending to buckle by the pressure of the water. They consisted of walings 12 inches square, laid horizontally along the sides of the caisson about 4 feet 3 inches apart between centres vertically, well strutted apart by struts also 12 inches square and placed about 7 feet 6 inches apart between centres. No concrete was deposited in the caissons until they were disconnected from the suspending-bars and were floating.

For the purpose of ensuring accuracy in the sinking of the caissons, guide-piles 12 inches square were accurately but lightly driven along the sides and ends of the sites of each. A caisson having been lowered into the water until it was found to be floating, the suspending-bars were disconnected and the caisson was eased against the guide-piles into true position for sinking. Concrete was then deposited in the caisson to a total height of about 26 feet 6 inches above the bottom of the cutting edge, this level being determined partly by consideration of the weight required for sinking on the basis of the experience gained with the land piers, and partly by the fact that it was a convenient level to begin building, the bottom of the brick pillars being just at the top of one of the series of cross bracings.

With the concrete work completed, compressed air was applied, and sinking was commenced by removing the excavations in the working-chamber. The caisson being partly sunk into the bed of the river without any excavations having been removed, the working-chamber was usually found to be partly filled with soft "slurry." This material was difficult to remove, and, as the water could not be separated from the mud and blown out with compressed air, the slurry had to be bailed out in buckets before excavating proper could be commenced. For some time, therefore, after the sinking of the caissons commenced, there was room for only two or three men in the working-chamber, and it was usually a fortnight before the full complement of men could be employed.

At the commencement of the excavation in the working-chambers of the river-pier caissons, the actual air-pressure with about 27 feet head of water from the cutting edge was about 12½ lbs. per square inch, that is, about ½ lb. per square inch more than appeared to be theoretically needed to balance the hydrostatic pressure. The deepest foundation of the bridge was in connection with river pier C, where caisson C 1 was sunk to a depth of 70 feet below high-water level. The highest air-pressure in the sinking of caisson C 1 was about 32 lbs. per square inch, which is about 1 lb. more than the theoretical pressure due to a head of 70 feet. In tunnelling under compressed air, the Author's experience has been similar. The actual intensity of pressure appears always to exceed slightly the theoretical requirement, presumably owing to loss by leakage.

In sinking caisson C1 the number of men in the working-chamber never exceeded six, for the excavations, being mostly in sand and gravel, were comparatively easily dealt with, so that there was little trouble in "getting," and frequently the men had to wait for the return of the bucket from the top. There was no difficulty in sinking this caisson, but for a time the progress was somewhat slow, by reason of there being only one air-lock in operation, the others being in use in connection with another caisson.

The maximum number of men employed in the working-chamber in the sinking of caisson C2 was four, and here again the material of the excavations was sand and gravel. Before the concrete work of caisson C2 was quite completed—that is, the concrete outside the working-chamber—the caisson canted badly owing to one end suddenly touching hard ground. An air-lock was immediately fitted, and sinking under air-pressure was commenced, with the object of righting the caisson, and this being readily accomplished, the concrete work was finished. From time to time during the sinking of this caisson the position was carefully checked, when it was found to tend to travel towards caisson C1, already sunk, the ground being evidently loosened in that direction. This tendency was corrected by sinking the caisson in a somewhat inclined position.

The material of the excavations in the sinking of caisson B1 was for a considerable depth largely muddy clay. After the water was driven out, the action of the compressed air on the clay made it difficult to excavate, so that reasonable progress in the working-chamber could be maintained only by having nine men constantly employed: even with this larger number the men on the top had sometimes to stop, and the cranes for disloading the buckets were frequently at a standstill. The progress of the sinking of this caisson was, therefore, comparatively slow, and to make matters worse, by reason of the cutting edge of one side of the caisson suddenly getting into sand and gravel, the cutting edge of the other side seemed to slip through the clay and the caisson canted. It was, however, very quickly righted again. At another time, the compressed-air supply was suddenly cut off during the progress of the sinking of this caisson, due to freezing of the pipes, but fortunately the cutting edge happening to be well embedded, there was no noticeable movement of the caisson. Following this, the main air-compressor broke down, so that the auxiliary compressor had to be brought into operation. The auxiliary compressor was not, however, able to maintain the required pressure, and the decrease of pressure resulted in the caisson gradually dropping about 2 feet, fortunately without detriment, as no canting took place.

While the sinking of caisson B 1 thus appeared to be attended with quite a chapter of small incidents, the sinking of caisson B 2 was altogether uneventful, and it was indeed marked by excellent progress, notwithstanding that the excavations were much in the muddy clay.

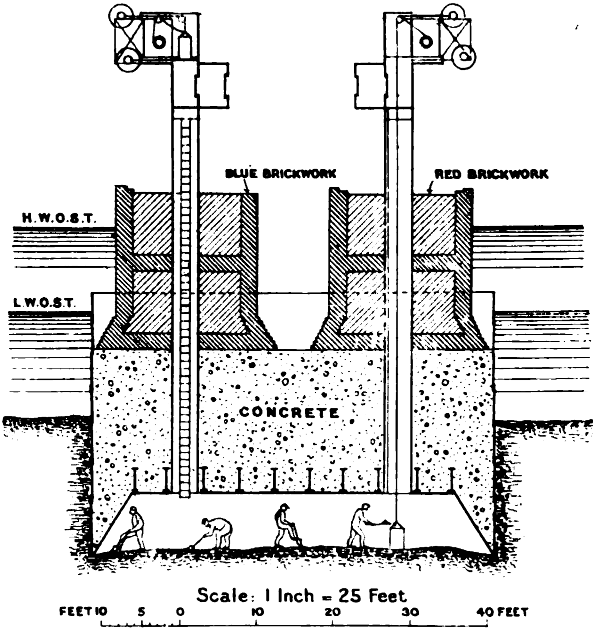
There being comparatively little surface friction in connection with the caissons of the river piers, the level of the excavations in the working-chamber was always kept about 15 inches above the level of the cutting edge. When the caissons reached their proper depths, the excavations in the working-chamber were taken down to the cutting edge in short lengths, the material being removed at the ends first, where the concrete was deposited and allowed to set in the form of a supporting block before the excavations of the centre part were removed.

The brickwork of the pillars was commenced as soon as deposition of the concrete over the working-chamber had been completed and the caisson had been levelled and firmly embedded in the bottom. It was continued during the progress of the sinking, so that, as excavating in the working-chamber proceeded and the caisson went down, the tops of the pillars were always maintained above high-water level; and as the brickwork reached the undersides of the air-locks, the locks were removed and replaced, after additional lengths of access-shaft had been added. The simultaneous excavating, building, and sinking is illustrated by *Fig. 12* (p. 34).

In some cases the top lengths of the access-shafting were temporary and were subsequently removed. Simultaneously with the building of the pillars, the excavations from the working-chamber were deposited in the caisson on the top of the concrete around the brick pillars for weighting purposes.

The granite work of the pillars was built similarly and simultaneously, so that the finishing of the sinking of the caissons of the river piers required much more care than that of the caissons of the land piers. The lowest granite course of the river piers is about 5 feet below low water, and in the building and sinking careful attention had to be paid to regularity in level as well as in line. The first caisson to be finished was caisson C 1, and owing to the fact that it showed no tendency to sink unduly quickly after the reduction of the air-pressure, the second course of granite below the moulded base-course of the several pillars was finished practically at the correct level. The line was also good, for when the sinking was completed the pillars were ascertained to be only 2 inches to the south and 3 inches to the west of their proper position. It therefore became a question of having the pillars of the other three caissons relatively correct in line and level, and the sinking and the further finishing was successfully carried out accordingly. In the case of caisson C 2 there was a difficulty in founding at the correct level relatively to caisson C 1. When the caisson had still 4 feet to go, the working-chamber was cleaned out and the air-pressure was reduced. As the caisson sank, observations were made with the level in order that the sinking might be properly governed, and timeously stopped, but in the course of sinking, the caisson began to cant, and this to such extent that the air-pressure had to be increased, and men had to be sent down to embank material against the low side with the object of righting it. Ultimately the caisson was founded satisfactorily, and when the sinking was

Fig. 12.



SINKING A CAISSON.

completed there had been three courses of granite built in each of the pillars.

The granite course immediately below the moulded base-course was in each of the pillars of the several caissons made a compensation-course, and, although its nominal thickness was 2 feet 3 inches, the actual thicknesses varied with the adjustment of the levels, but only to a slight extent.

The concrete was deposited in the working-chambers of the river-pier caissons in a manner similar to that adopted in the case of the caissons of the land piers already described. It being of vital importance, however, that the caissons of the river piers should be maintained at true level after the sinking was finished, the working-chambers were not at once wholly cleared out. They were cleared out in small sections.

Taking caisson B 1 as an example of the loads, when pitched in the river-bed with the concrete work partly executed, it weighed 866 tons, and when the concrete work was finished and sinking under compressed air was commenced, the total weight was 2,420 tons. When the sinking was finished, the aggregate weight of the steelwork, concrete, brickwork and granite was 3,854 tons, and this was further increased to a grand total of 4,597 tons by the deposition of the concrete in the working-chamber and access-shafts. The gross load (due to the dead weight and live load) at the bottom of caisson B 1 was calculated to be 10,446 tons.

Under the contract it was specified that when the masonry of the substructure of each pier was completed, the foundations should be tested by applying such weight as would temporarily load the bearing-area to an extent 25 per cent. greater than the permanent load. The character of the foundations as revealed by the actual excavations was, however, thought to be so satisfactory that it was decided that this testing need not be carried out—a decision which was subsequently ascertained to be justified. Much money was thereby saved, as about 6,000 tons of pig-iron or steel rails would, for example, have been required for caisson B 1, and, not to speak of the practical difficulty and the cost of getting such testing-materials to and from the site, the contract-prices were 5*s.* per ton for merely applying the load and 5*s.* per ton for afterwards removing it.

Rate of Progress of the Excavations in the Caissons.—The tabulated statements given in Appendix V show the character of the geological deposits through which the main caissons of the several land and river piers were sunk and the time taken to sink each caisson, etc., and they afford a means of comparison of the rate of progress of the excavations in strata of somewhat varying character. The stated volumes of excavation are based on notes taken in respect of depths near the bottom of the foundations, that is, during the period more immediately prior to completion of the sinking, when the work of excavating in each caisson was in full and active operation. For ready comparison, the final results are brought out in fractions of a cubic yard per man per hour. The variation in the results is due partly to the different character of the strata and partly to the fact that sometimes there were two locks in operation in one caisson and sometimes only one, while at other times there were three.

The average rate of excavation in the working-chambers of the caissons was at the rate of 0·47 cubic yard per man per hour in the case of the land piers, and 0·60 cubic yard per man per hour in the case of the river piers, so that the average over all may be approximately stated at rather more than ½ cubic yard per man per hour.

As already indicated, the number of men in the working-chamber varied with the size of the caisson and the special circumstances attending the sinking. In the case of the land piers six men were, as a rule, constantly employed down below, but in the working-chambers of the river-pier caissons there were sometimes as few as four and as many as nine. The men were double-shifted and each gang was 12 hours on the work at a time, but there being two meal-hours intervening, they only actually worked 10 hours per shift.

In the sinking of caisson B2, which was one of the smaller of the main caissons, with two air-locks in operation, the following men were employed, at the wages stated:—

One miner at	7*s*. per shift.
Three miners at	6*s*. „ „
Two lockmen „	3*s*. 6*d*. per shift.
„ enginemen at	5*s*. per shift.
„ crane men „	6*s*. „ „

In addition to these men there was a man in charge of the steam-boiler in connection with the engines at 5*s*. per shift and another in charge of the air-compressor at 5*s*. per shift, and, of course, as well as foremen there were other men employed in the general work.

Frictional Resistance of Caissons.—In the course of the sinking of the caissons tests were made with the object of ascertaining the bearing value of the frictional resistance resulting from the lateral pressure of the earth on the sides and ends of the caissons. In the case of the caissons for the river piers, owing to the softer character of the material of the excavations and the possibility of the roof of the working-chamber bearing down and crushing the men inside, it was not considered expedient to excavate entirely and clean out down to the cutting edge. And there was the further difficulty of the uncertain weight of the excavated material deposited on the caisson during the sinking; so that the results of certain tests in connection with the river piers were considered to be of comparatively little value. The circumstances of the caissons of the land piers, however, were more favourable, the material of the excavations being stiff and the surface friction on the sides and ends of the caissons so considerable that on occasions the pressure of the air had to be reduced to induce the caissons to move. It was therefore possible to make more accurate tests in connection with the caissons of the land piers, the results of which showed the value of the surface friction to range from 3¼ cwt. per square foot to 4¾ cwt. per square foot, the average of five careful observations being about 4¼ cwt. per square foot. The strata were mostly sand and gravel, and the embedded surfaces of the caissons were in contact to a depth of about 40 feet. In making the observations on the caissons of the land piers, the excavated material was entirely removed from the working-chamber as low as the cutting edge, and the edge was undercut as much as was possible. The men were then withdrawn from the working-chamber, and the air-pressure was reduced, when the caisson ultimately began to sink; as sinking commenced and continued the intensity of the air-pressure was noted on the gauge at the material-lock, and this pressure, which varied between 5 and 15 lbs. per square inch over the series of observations was discounted in arriving at the results. The gross surface friction being equal to the gross weight of the caisson and its loading minus the gross pressure of the air on the roof of the working-chamber, the unit frictional resistance was, of course, the quotient resulting from division of that difference by the area of embedded surface. In most circumstances, the certainty of there being some resistance by the cutting edge should be taken into account, so that in the safe-load calculations for foundations in such ground the bearing-value of frictional resistance might be stated at about 3 cwt. per square foot. In the determining of bearing-areas frictional resistance is, however, more of academic than practical interest, and the expediency of depending on it and giving it value in the safe-load calculations is extremely doubtful. But it is an important factor in the problem of the sinking.

UNIT PRICES FOR SUBSTRUCTURAL WORK.

The items and unit prices of the substructural work of the river piers were the following:—

Clearing and levelling bed of river and pitching caissons thereon	15*s.* per square yard.
Excavations in and sinking caissons from level of bed of river downwards . . .	20*s.* „ cubic „
Wrought steelwork in permanent caissons .	£15 per ton.
Concrete in caissons.	22*s.* 6*d.* per cubic yard.
„ „ working-chambers	28*s.* 6*d.* „ „ „
Red brickwork in hearting of pillars . . .	22*s.* 6*d.* „ „ „
Blue „ „ through courses and facing of pillars	43*s.* „ „ „
Granite ashlar in facing of pillars below base course with chisel drafted margin and rock face	6*s.* 3*d.* „ „ foot.

Based on these unit prices, the aggregate unit cost of the substructural works—namely, the excavations and all the kinds of masonry in the caissons and pillars, including the masonry of the pillars up to low-water level—was on the average £2 15*s.* 6*d.* per cubic yard of the earth and water permanently displaced, or £2 10*s.* 6*d.* per cubic yard of such displacement if the masonry of the pillars is included only up to a level 5 feet below low-water level, that is up to the level where the brick facing ends and the granite ashlar facing begins. These costs, while inclusive of all labour at the site, do not include the cost of the steelwork of the caissons, because the caissons may be described as being of the character of service work and merely means towards an end. It would, however, be proper to include the cost of the steelwork of the caissons, and if this were done, the foregoing unit costs would need to be increased by about £1 1*s.* per cubic yard. In order to arrive at the absolutely correct cost, a portion of the general charges under the contract, such as the costs of temporary staging, setting out, watching and lighting, and several other contingent works, for which there are special prices in the contract-schedule, should also, of course, be included; but an addition of 5 per cent. would more than cover such general charges. In estimating the cost of such work for Parliamentary purposes, the Author would adopt about £3 15*s.* per cubic yard of the earth and water displaced by the permanent works, as an inclusive unit price to pay for everything submerged.

The Superstructure.

General.—As the steel superstructure of the part of the bridge over the river and quays consists of a series of main longitudinal girders carrying transverse flooring on the top flanges, there is no complexity in design and the loads are transmitted very directly.

The superstructure over the street on either side of the river is also simple, the steelwork being composed of longitudinal and transverse plate girders with buckled-plate flooring. The cross girders are attached to the webs of the longitudinal girders, a connection which, having regard to maintenance, cannot perhaps be described as ideal ; but they are also afforded support by the stiffeners of the main girders, which are made to act as stools. Experience in maintenance suggests that it is very desirable to have the cross girders of a railway-bridge resting directly on the top flange of the main girders ; but the levels of the street spans of the Clyde Bridge precluded the possibility of giving the cross girders any better support than the attachment to the webs provides. The buckled plates of the flooring of the street spans, although stamped and buckled to about the usual size, are in lengths of 14 feet; they are closely riveted to the cross girders at their ends only, and are merely attached to the intermediate cross girders by a few stitching rivets.

The main girders of the river spans are alternately supported directly over the pillars of the piers and midway between the pillars of the piers, pivoted on special cast-iron bearings which in their turn rest on steel lintel-girders spanning the spaces between the pillars.

By reason of the fan shape of the bridge at either end the longitudinal girders are not parallel in line, and in plan they form tapering bays. They are spaced at distances apart varying from 11 feet 10 inches in the south river span to 15 feet 10 inches in the north river span. There being a limit to the piling of plates in the flanges of girders, a system of wider spacing of the longitudinal main girders with cross girders between them was quite prohibitive, even if there could have been satisfactory attachment of the cross girders to the main longitudinal girders. With wider spacing of the main longitudinal girders the sectional area of their flanges would have had to be very great, as their depths are very shallow. The main longitudinal girders are indeed rather unsatisfactory in respect of the ratio of depth to span. The length from centre to centre of the bearings of the girders is 194 feet in the case of the centre span, and 151 feet 3 inches and 173 feet 3 inches respectively in the case of the north and south river spans. Measured between the centres of gravity of the flanges, the girders of the centre river span are 15 feet 9 inches deep, and of the north and south river spans each 13 feet deep. The ratio of depth to span is therefore 1 to 12·32 in the case of the centre span and 1 to 11·63 and 1 to 13·33 in the case of the north and south river spans respectively. But the best was done, and the shallow depths and indifferent proportions were unavoidable on account of the limited depth available for construction.

Each longitudinal main girder at the south end of the bridge is calculated to carry the load of a single line of rails and about the load of one and one-quarter line of rails at the north end; and these loads, combined with the heavy superincumbent dead load, necessitated considerable flange-area. The girders are therefore comparatively heavy. Except the extreme outside girders, which take less load, each main girder of the centre span weighs about 220 tons and each main girder of the north and south river spans weighs about 143 tons and about 162 tons respectively, while the weights of the outside girders of the three spans are 133 tons, 84 tons, and 111 tons respectively.

The exceptional dead load on the bridge is largely due to the superstructure being entirely covered with the ballast of the permanent way, and the live load is of course that resulting from the Company's latest and heaviest class of locomotives. These locomotives have 18¾ tons on the driving-axle, and when fully equipped, engine and tender combined weigh about 130 tons, distributed on a wheel-base of about 57 feet. The length over the buffers is 65 feet 6 inches, and having regard to the spans of the bridge and the certainty in the vicinity of a terminal station of there being exceptional loads due to several locomotives being at times coupled together, the equivalent uniformly distributed live load on all the spans of the bridge, although they are not the same, was assumed to be 2 tons per lineal foot of single line of railway, a live load which, if there is to be any regard at all for the economics of railway-maintenance, it should not in the future be allowable or necessary to exceed. From this point of view, indeed, locomotives are already more than heavy and powerful enough.

The steel was made by the Siemens-Martin open-hearth acid process, and it was specified that the tensile strength should not be less than 28 tons and not more than 32 tons per square inch of original section, with elongation of at least 20 per cent. in a length of 8 inches. By reason of the practical inconvenience which would have resulted from cutting plates and bars in the bridge-building yards and taking specimens for testing there, the specimens were taken and tested at the steelworks, a practice which, although the more common practice, perhaps leaves something to be desired. It would be better to take the specimens for testing from the material when delivered in the bridge-building yard; but practical difficulties might present themselves, and in any case such an arrangement would require united action on the part of engineers and would necessitate a standard specification.

The main girders were designed to have a working-stress of about 6 tons per square inch, thereby giving a factor of safety of about 5. In the case of the flooring, however, the working-stress was fixed at 4 tons per square inch; for although, on account of the bridge being near the station and the speed of trains being consequently low, it was not anticipated that the impact would be great, yet the girders being so directly over the river, it was thought that the tendency to corrosion might be considerable, and full allowance for deterioration was therefore made.

The practice in bridge-calculations in respect of concentrated maximum live loads and equivalent uniformly distributed loads, and the allowable stresses in the several parts of the steelwork of railway underbridges, might with much advantage be standardized; and it would be more advantageous still if a central authority, such as the Engineering Standards Committee, were to standardize the practice in respect of the dead and live loads to be assumed in calculating the strengths of the several classes of bridges carrying urban streets and country roads respectively.

For the purposes of design and payment the weights of the steel and iron of the Clyde Bridge were assumed to be as follows:—

Cast steel	492 lbs. per cubic foot.
Wrought steel	490 ,, ,, ,,
Wrought iron	480 ,, ,, ,,
Cast iron	448 ,, ,, ,,

although the true weights of these are said to be fractionally less; and with no reduction for rivet-holes, 5 per cent. extra on the weights of the wrought-steel plates and bars was allowed for rivet-heads, a figure which perhaps is also somewhat high.

The Main Girders of the River Spans.—A main girder of the centre river span is shown by Figs. 13 and 14, Plate 2. It is typical of all the girders, which are of the Linville or **N** type of lattice girder.

While the scantlings of the main girders of the several spans of course vary with the stresses, the girders are identical in their design and construction. Although the girders differ in total length, the panels formed by the vertical members of the web-bracing are the same in each girder of the several spans, except the end panels, which vary. The vertical members of the girders of the centre span are 8 feet apart from centre to centre, while those of the north and south river spans are 7 feet and 6 feet 8 inches apart respectively.

It will be seen that the top and bottom booms of the girders are each built up with a series of horizontal plates, two vertical plates and four main angle-bars; and for the purpose of stiffening the top boom a continuous angle-bar is attached to the lower outside edge of each vertical plate. With the object of having a moderate number of piled plates the flanges were made 4 feet wide; but even with this width the thickness of the plates at the middle is in the aggregate $5\frac{5}{16}$ inches, made up of nine plates varying from $\frac{3}{8}$ inch to $\frac{5}{8}$ inch in thickness.

The vertical plates of the booms are 2 feet deep, and vary in thickness from $\frac{1}{2}$ inch at the centre of the girder to $\frac{3}{4}$ inch at the end, depending on the bearing area required by the rivets at the connections. The four main angle-bars are each 5 inches by 5 inches by $\frac{5}{8}$ inch, and the stiffening angles of the top boom are each $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{1}{2}$ inch. The girders were given 1 inch of camber for every 60 feet of span.

The struts of the web-bracing are of box section, built up of channel-bars varying from 10 inches by $2\frac{9}{16}$ inches by $\frac{1}{2}$ inch to 12 inches by 3 inches by $\frac{1}{2}$ inch, strengthened where required by plates of various scantlings, except near the ends of the girder where the box section is built up of plates and angles. The struts are laced with flat bars varying from 3 inches by $\frac{7}{16}$ inch at the ends of the girders to 3 inches by $\frac{3}{8}$ inch at the centre. The ties of the web-bracing are in the form of four flat bars, pairs being riveted to the vertical plates on either side. The bars vary in section as shown, and where the stresses alternate the inclined members are designed as struts, and consist of two channel-bars 9 inches by $3\frac{3}{16}$ inches by $\frac{1}{2}$ inch, stiffened at intervals with 9-inch by $\frac{1}{2}$-inch plates.

The centre-lines of the struts and ties of the separate systems of bracing intersect at the top and bottom respectively at points corresponding with the centre of gravity of the flange at each apex.

The several girders are braced together at the top and bottom booms at intervals of about 24 feet with diagonal bracing in a vertical plane, consisting of horizontal channel-bars and diagonal tee-bars, and the bottom flanges are further braced horizontally by diagonal bracing in the form of angle-bars. For the purpose of stiffening the booms at the vertical diagonal bracings, diaphragms are inserted between the vertical plates. The flooring, of course, braces the girders at the top.

The rivets through the horizontal plates of the flanges are 1 inch in diameter and at 4 inches pitch, except those near the centre-line of the flanges, which, being merely stitching rivets, are 1 inch in diameter and at 8 inches pitch. All the other rivets are $\frac{7}{8}$ inch in diameter and at about 4 inches pitch. The riveting of the girders was executed by hydraulic and pneumatic power, depending on the position of the rivets, with distinct preference for the hydraulic power where many plates were piled. There was no hand-riveting. The ties and struts of the web-bracing are not riveted together where they cross, so that they act quite independently. To prevent flapping, however, the pairs of flat bars forming the ties are held together near their middle with two bolts which pass through hollow distance-pieces fixed between the pairs of bars.

The Lintel-Girders.—The lintel-girders (Fig. 5, Plate 1, and Figs. 17, Plate 2), by which the main girders are supported, span from pillar to pillar of each pier, and are bolted to granite blocks by four 1½-inch-diameter bolts at each bearing. They are box girders 4 feet deep, with flanges 3 feet 6 inches wide, and their effective spans vary from 19 feet to 25 feet. The details of their construction are clearly indicated in the above-mentioned Figures. The main girders rest over the centres of the lintel-girders and over the ends of adjoining lintel-girders at their bearings. It is estimated that the load at the centre of each of the lintel-girders of the river piers is about 458 tons: and, having regard to this, there is a stiff diaphragm at the centre of the lintel-girders connecting the three web-plates, and similar diaphragms at each bearing on the pillars. With a view to distribution of the load on the pillars of the piers, the lintel-girders have enlarged bearing-plates riveted to their bottom flange, and they rest on a grillage composed of 6-inch by 4½-inch by 20-lb. rolled beams spaced about 10 inches apart and riveted to a 1-inch plate, which rests on the granite, as shown by Fig. 17, Plate 2. The bottom flanges of the lintel-girders were machined at the bearings after the girders were built, and the space between the rolled beams of the grillage is filled with granolithic concrete, finished quite smooth at the top in order to ensure true and uniform bearing of the lintel-girders.

The Bearings.—The bearings for the main girders (Figs. 18, Plate 2) rest on the top of the lintel-girders. There are two kinds of bearings, one for the expansion and the other for the fixed end of the girders. The several parts of the bearings are of cast iron, specially machined at their surfaces of contact. The rocker of the expansion-bearing is machined at the top and at the bottom to a radius of 7½ inches, and fits into the semicircular surfaces of the top and bottom saddles, which are machined at the same radius as the rocker, for a circumferential length equal to about two-thirds of the semicircle, which is the measure of the bearing at the place of contact. The expansion-rockers under certain of the girders were set vertical when the temperature was normal, but in extremes of temperature others were set inclined from the vertical to the extent of about $\frac{5}{32}$ inch for each 10° F. variation of temperature above or below the normal, which was assumed to be about 55° F.

The fixed bearings are merely deflection-knuckles similar in form to the upper portion of the expansion-bearing. The upper part of the knuckle and the under part of the top saddle are machined to a radius of 7½ inches, and have bearing surfaces similar to the bearing surfaces of the expansion-bearings.

The expansion and fixed bearings are similar in size and the fastenings are also identical. The main girder is bolted to the top saddle, and the bottom saddle is bolted to the lintel-girder and held longitudinally by lugs cast at the side. Pairs of expansion-bearings rest on the north river pier, and pairs of fixed bearings rest on the south river pier.

The bearings afford good points of support, and observation shows that the girders appear to adapt themselves readily enough to expansion and contraction with changes of temperature and to deflection under the loading. Similar satisfactory results might, however, have been attained with simpler bearings, recent experience in maintenance having shown that long-span girders of certain bridges and roofs had not apparently suffered, although the special bearings on which they rested had rusted and were immovable. But in these cases there was space for the girder to slide freely.

The Flooring.—With a view to lightness of dead load, the flooring of the main part of the bridge is Hobson arched-plate flooring (Figs. 13, 14 and 20, Plate 2). It is laid transversely and rests on the top flanges of the main girders, spanning spaces between these girders varying from about 8 feet to about 12 feet. It is 12 inches deep and is built up with a curved plate ½ inch thick and a tee-bar 7 inches by 3½ inches by ½ inch, strengthened where the spans are wider by the substitution of two angle-bars for the tee-bars, and by the addition of an arched plate round a part of the circumference. The rivets are ⅞ inch in diameter, and at 4 inches pitch, and they were put in by hydraulic pressure, a special machine having been made for the purpose.

The flooring has a bearing of 12 inches on the top flanges of the main girders, so that there is a space of about 2 feet over the main girders between the ends of the several bays of flooring. This space adapted itself to drainage purposes. The flooring is not packed up to the level throughout the length of the main girders, but follows the flange-plates and the camber of the main girders, so that the surface of the flooring near the ends of the main girders at the piers is at least 6 inches lower than the surface at the centre of the main girders. This was also taken advantage of in the drainage, as it lent itself to grading in the work of waterproofing.

As the main girders are not parallel, the flooring could not be laid normal to them all. Each arched plate of the flooring is, however, cut square at the ends, and the skew is taken up by a dished plate inserted at each end, to which the flooring is riveted, the dished plate being in its turn riveted to an angle-bar, which runs the whole length of the main girder.

At the ends of the main girders over the piers the flooring is stopped and finished with an ordinary plate cross girder, which, like the flooring, spans the space between the main girders. There is thus a space longitudinally over the piers, between the flooring cross girder of the one span and the flooring cross girder of the next span. This space, which is 16 inches wide, is covered by a plate 1 inch thick. At the fixed ends of the main girders the plate is bolted to both the cross girders and thus the flooring is made continuous over the pier. At the expansion ends, where the main girders rest on the movable bearings, the 1-inch plate is bolted to only one of the cross girders and is allowed to slide on the other cross girder, or rather on part of a plate, known as a splash-plate, bolted to the cross girder, as shown by Fig. 19, Plate 2.

The splash-plate is for the purpose of preventing water from getting on to the steelwork opposite, for here there is a drainage-outlet, and water from pipes laid longitudinally along the top of the main girders between the bays of flooring flows into a gutter, which lies, between the ends of the girders, along the tops of the piers for the whole width of the bridge.

The Parapet-Girders.—The parapet-girders, Fig. 15, Plate 2, are lattice girders, with six systems of duplicate bracing. The booms are each built up of a series of horizontal flange-plates, two vertical plates, and two angle-bars. The inside members of the web-bracing are flat bars, but the outside members are channels cut to pass each other at their intersection, where they are covered with a cast-iron rosette.

The parapet-girders are placed at a higher level than the main girders, the bottom of the former being about the same level as the top of the latter. They are 10 feet 4½ inches deep, and their top flanges are 7 feet above rail-level. They are practically self-supporting, but they are strengthened by bearing on a series of steel brackets of curved form, which are attached to the outer main girders. They are placed 6 feet 6 inches out from the centre of the main outside girders, in order that there shall be 7 feet of clearance from the rail, the effect of which is also to allow of a footpath being formed on either side of the bridge, about 5 feet wide, for the use of signalmen and other servants of the railway-company. In fixing the height of the parapet-girder at 7 feet above rail-level, and its position at 7 feet from the nearest rail, regard was had to the possibility of trains being stopped on the bridge, and passengers, thinking the train was in the station, attempting to alight.

THE ERECTION OF THE MAIN GIRDERS.

The main girders were constructed on the temporary staging across the river and on extensions of it over parts of the quays. The struts of the main bracing, together with the connecting plates, were built up in the bridge-yard, and so conveyed to the site, but all the other parts of the girders were taken to the site in the form of prepared plates and bars, and were built up and riveted there. In erection, the girders were supported on camber-blocks or wedges, and on a series of timbers laid transversely 7 feet apart and built up to a height sufficient to allow of the workmen conveniently passing underneath the girders for riveting and other purposes. At first the work of building the girders had to be carried on entirely at the level of the staging, but as girders were completed, use was made of some of them for erection of the others of the same span, by placing cranes and having bogie-roads on them and otherwise. When the riveting was completed, the camber-blocks were removed and the several girders were eased into position and lowered to their permanent bearings by means of hydraulic jacks.

THE STEELWORK OF THE SIGNAL-CABIN.

The signal-cabin, which is 106 feet long and 16 feet wide, is situated between the new and the old bridges, the floor being 15 feet above rail-level. It is carried on an overhanging steel frame attached to the parapet of the new bridge, and is otherwise supported by a series of brackets attached to the main outside girder

Access to the signal-cabin is got by means of a gangway, 5 feet wide in the clear, erected across the bridge on the line of the north river pier, as shown on Figs. 2 and 4, Plate 1. The main girders of this gangway are of the Vierendeel type, being so designed in order to afford a clearer view than could be got through the bracing of an ordinary lattice girder, it being the intention to use the gangway as a platform from which men will direct the movements of trains. The span of the gangway is 120 feet and the girders are 10 feet deep at the centre and 5 feet deep at the ends. The scantlings are as shown in Figs. 16, Plate 2. Access to this gangway is got by means of a spiral stair constructed inside the granite pilaster of the bridge at the north river pier.

The Painting.

Except the caissons and the upper surface of the plate flooring, the steelwork of the bridge was first coated with boiled linseed-oil and afterwards painted with four coats of paint. The plates and bars were coated with oil before being put together, and, after being riveted up, they received a priming coat of red and white lead mixed with pure linseed-oil. The priming coat was put on in the bridge-yard before there was any corrosion, and after all loose scale and dirt had been carefully removed and the steel was made thoroughly clean; and the remaining three coats, also composed of leads and linseed-oil, were put on after erection of the structure. In order to ensure the painting being properly executed, the several coats were tinted to different shades, the finishing coat being of a light stone colour.

Experience has shown that lead paint is the best preservative of steel and iron; but the expediency of first coating the metal with oil is somewhat doubtful, although the possibility of corrosion setting in before the paint has been applied should be absolutely precluded.

With a view to provide permanent facilities for inspection and future painting and other maintenance work, light gangways have been erected along one side of each of the main girders at the level of the bottom flanges: these gangways are connected with each other by means of a transverse gangway at the middle of each span. They are in the form of steel brackets attached to the main girders and have a timber floor and a light iron handrail. Access to the gangways is obtained from the footpath at rail-level on either side of the bridge by means of man-ways and iron ladders.

The Waterproofing.

Apart from a statutory obligation on the Company to have the superstructure of the bridge thoroughly watertight and thereby preclude the possibility of water dripping through on to the streets and quays below, the importance of proper waterproofing was recognized from the point of view of the preservation of the structure.

In the design of the waterproofing it was made an imperative condition that the surface to be protected should be properly prepared to receive the waterproofing by being carefully graded and steeply sloped in all the necessary directions. As a means towards this end it was necessary to lay a "levelling up" layer to form a bed for the waterproofing, and this layer also acted as a preservative covering.

There was much consideration of the character of the waterproofing, and experience having shown the futility of waterproofing with artificial asphalt even if "reinforced" with layers of tarred brown paper or bagging or other textile material, and regard being had to elasticity and rigidity and the influence of temperature and vibration, it was decided to waterproof the steel structures with natural rock asphalt, and those of masonry with cement rendering, the graded bed or preservative covering underneath being either cement concrete or bituminous concrete. The waterproofing was therefore made to consist of two layers, a lower or preservative layer and an upper or protective layer.

In the execution of the waterproofing work the upper surface of the Hobson flooring of the bridge was first of all brushed and cleaned and coated all over with boiling bitumen. The hollows at the joints between the arch-plates of the flooring were then carefully plugged in the bottom with Seyssel rock asphalt to a depth of about 3 inches. Above this plugging, the arched plating was haunched with bituminous concrete (a conglomerate mixture of broken stone and boiling pitch) to the level of its upper surface, and to some extent above it at the centre of each of the bays of flooring between adjoining longitudinal girders, where a summit was formed and the surface was so graded as to ensure there being a good cross fall on either side towards the channel on the top of each longitudinal girder which formed itself between the ends of each of the bays of flooring. These channels were shaped in the bottom by filling with fine concrete in cement to such a semicircular form as would properly receive drainage-pipes, Fig. 20, Plate 2, and they were so graded in level longitudinally as to give a sufficient fall towards either end.

With the surfaces of the flooring and the channels thus prepared with the preservative covering, they were protected by being waterproofed and made impervious with an all-over layer of Seyssel rock asphalt ¾ inch thick, the finished surface of which is somewhat similar to that of the paving of some of the streets of London.

In each of the semicircular channels on the tops of the main girders between the ends of the bays of flooring, there are 6-inch-diameter cast-iron pipes perforated on their upper surface, and having inspection-chambers about 50 feet apart. The pipes, which have open joints, are haunched with concrete at their undersides and covered over with broken stone, so that the water from the graded surfaces of the respective bays of flooring flows freely into them; and being connected with gutters laid longitudinally along the piers between the ends of the main girders, they ultimately have their outfall in the river.

The waterproofing of the buckled-plate portions of the flooring of the parts of the bridge over the streets was somewhat similarly executed, but, instead of bituminous concrete, fine cement concrete was used as a preservative and making-up covering, and, being laid to the level of the top of the tee-bars and carefully graded, was also covered all over and made impervious with a layer of Seyssel rock asphalt ¾ inch thick. The water is led to down pipes which discharge into drains having connection with the public sewer.

In order to preclude the possibility of injury by the angular ballast of the permanent way and the picks and shovels of the surface-men, the waterproofing throughout its entire area is covered with a layer of very finely crushed whinstone, about 2 inches deep, on which the ordinary under-ballast is laid directly.

The bottom booms of the main girders were waterproofed with a filling of bituminous concrete shaped to circular form and laid to proper fall, the outfall being through certain vacant rivet-holes.

From the point of view of the economics of maintenance, waterproofing work does not appear to have always received that consideration and care which its importance imperatively demands. A preserving layer of concrete covered with a protecting layer of natural rock asphalt is the best waterproofing that can be executed; but the primary requirement in all waterproofing work is to provide plenty of "fall."

The Elevation.

The Company were under statutory obligation to the City Corporation in respect of the appearance of the bridge, so that in the design special regard was given to the perspective, which was carefully studied by means of a model. Endeavour was made to secure effect by the introduction of pleasing lines and by contrast in light and shade, rather than by added ornament; and without sacrificing economic principle, the several parts of the structure were treated accordingly. The result is indicated in Fig. 21, Plate 2.

The pillars of the piers taper 1 in 72 upwards from a moulded base-course, which is situated about high-water level. The outer pillars are built up to the level of the top of the main girders and finish there with a heavily projecting capital, on which the parapet girders are borne. Above the capitals the outer piers are continued upwards in the form of pilasters of semi-octagonal shape and of otherwise varied outline, and they are finished at the top with massive moulded copings. The masonry of the pillars is light grey granite ashlar of rock-face dressing below the level of the base-course and of fine-axe dressing above.

The parapet-girder was designed to give the appearance of mass, as well as continuity, and, with this in view, it is constructed in the form of a box girder having numerous horizontal lines at the top and bottom flanges. These lines are provided by the members of a cast-iron moulded base-course and a cast-iron moulded coping, simple in their detail, but with heavy projections and resulting light and shade. It was thought that the appearance of the bridge might also be improved by having the main girders deeply in shadow, and this was secured by placing the parapet and main girders in different vertical planes. The contrast between the main and the parapet girders is further accentuated by the web-bracing of the parapet-girder being made different from the web-bracing of the main girder. The latter with its heavy loads, even in appearance, calls for substantial vertical struts, but as the former is more in the nature of a fence, it was thought that from an æsthetic point of view, its requirements might better be met by having multiple systems of comparatively light inclined latticing with the bars covered at their intersections by rosettes of cast iron.

In the case of the river piers, the pilasters are carried up to a considerable height above the parapet-girders. This was done with a view to lead the eye upward, and in the perspective to emphasize the masonry and tend to dispel, or rather render less noticeable, the difference in level of the bearings of the main girders on either side of the several piers. It was thought at first that the broken line and want of continuity of the underside of the superstructure, due to the difference in level of the main girders of the several spans, might be detrimental æsthetically, but this possible defect in the appearance can be said to have been successfully corrected in the perspective by the adoption of the idea described. With the object of further enhancing the appearance of the bridge and relieving the angularity, its lines have been softened by the introduction of a series of curved brackets attached to the outside main girders. The parapet-girders rest on these brackets, and in the perspective the brackets have the appearance of continuously connecting the parapet and main girders by a line of pleasing curvature.

UNIT PRICES FOR SUPERSTRUCTURAL WORK.

The unit prices of certain of the superstructural works were the following:—

Masonry.

Brickwork in hearting of pillars of piers of red bricks	22*s.* 6*d.* per cubic yard.
Granite ashlar in bearing blocks of main girders, fine axed	7*s.* 9*d.* per cubic foot.
Granite ashlar in facing of pillars of piers below base-course with chisel-drafted margin and rock face	6*s.* 3*d.* ,, ,, ,,
Granite ashlar in moulded base-course of pillars of piers, fine axed	11*s.* 8*d.* ,, ,, ,,
Granite ashlar in shafts of pillars of piers, fine axed	7*s.* 10*d.* ,, ,, ,,
Granite ashlar in capitals of pillars of piers and bases, dadoes and capitals of pilasters heavily cut and moulded and fine axed	16*s.* 1*d.* ,, ,, ,,

Steel and Iron Work.

Wrought steelwork in open web main, plate main, box lintel, lattice parapet and other girders	£13 17*s.* 6*d.* per ton.
Wrought steelwork in curved brackets under lattice parapet girders	£13 17*s.* 6*d.* ,, ,,
Wrought steelwork in Hobson flooring	£13 17*s.* 6*d.* ,, ,,
Wrought steelwork in buckled plate flooring	£13 17*s.* 6*d.* ,, ,,
Cast ironwork in fixed and expansion bearings	£19 per ton.
Cast ironwork in moulded plinth cornice and coping of parapets	£18 ,, ,,

Waterproofing.

Bituminous concrete in haunches of and over the Hobson flooring	39*s.* 9*d.* per cubic yard.
Fine cement concrete (5 to 1) over the buckled plate flooring	24*s.* 9*d.* ,, ,, ,,
Seyssel asphalt ¾-inch thick covering the bituminous concrete over the Hobson flooring and the fine cement concrete over the buckled plate flooring	4*s.* 5*d.* per square yard.

These are the prices of the several items of work as stated in the contract-schedule, but, as in the case of the cost of the substructural works, if they are used for estimating total costs, a percentage should be added to cover the general charges which are common to contracts of this character.

THE TESTING.

For testing purposes, the bridge was loaded with locomotives in such a manner, and to such an extent, as to bring the greatest possible weight on to individual main girders. Nineteen engines were used, which were of the Company's heaviest class, their aggregate weight being about 1,167 tons. They were so placed on the lines of rails directly over the girders, and immediately adjoining the girders, as to load the main girders of the north river span, the centre river span, and the south river span, with a uniformly distributed weight to the extent of about 330 tons, 400 tons, and 300 tons respectively. The maximum deflections under these loads were about $\frac{5}{8}$ inch, $\frac{7}{8}$ inch, and $\frac{3}{4}$ inch for the north, centre, and south river spans of 151 feet, 194 feet, and 173 feet respectively; and these results were considered satisfactory. The lintel-girders were also tested, and, while there was indication of deflection, it did not in any case exceed $\frac{1}{16}$ inch. But with all the loading there was no indication of settlement of the foundations.

RAISING THE OLD BRIDGE.

Having regard to the levels of the extended station and of the approach-lines on the new bridge, it was necessary to raise the approach-lines on the old bridge. The old bridge has five spans which over the river vary from 151 feet to 194 feet, and it carries four lines of rails on rail-bearers, and cross girders resting on two outside main longitudinal girders 20 feet deep, which have their bearing on cylindrical granite pillars. Calculation having shown that it would be inexpedient to put any additional load on the old bridge in the raising of the rail-level, it was determined to raise the superstructure bodily. It had to be raised 1 foot 4 inches at its south end, and 3 feet at its north end, but with the bridge temporarily closed for traffic, the work was readily accomplished by means of hydraulic jacks applied to temporary steel brackets bolted to the end plates of the two outside main longitudinal girders of the several spans, purchase being got from the cylindrical pillars of the piers.

The Strength of the Iron of the Old Bridge.

It was thought desirable to ascertain if the wrought iron of the girders of the old bridge had suffered from the effects of the strains and exposure to which it had been subjected in its life of 27 years. Tests were therefore made on pieces of an angle-bar member of one of the struts of the web-bracing. There was very little variation in the results of the tests, and the breaking-weights were ascertained to average 23½ tons per square inch, the limit of elasticity 13½ tons per square inch, and the elongation on a length of 8 inches about 10 per cent. Two bending-tests were also made, with the result that pieces bent round a diameter of about 1 inch merely showed the slightest indication of fracture. The wrought iron had been kept well painted, and, there being little if any diminution of section and, as the tests showed, no deterioration in quality, the metal was considered to be still in a remarkably satisfactory condition.

The locomotives of the day are already found burdensome enough, and were it not that sooner or later those of the future may prove to be too fatiguing, the wrought iron of the old bridge, having regard to the likelihood of less corrosion, might last as long as the steel of the new, notwithstanding that the wrought iron has already been 27 years longer in existence. From the point of view of true economy, therefore, it is suggested in this connection that, taking everything into account, there may have been some inexpediency in so drastically discarding wrought iron for mild steel in bridges and other structural work.

The Executive.

The new bridge was designed and constructed under the direction of the Author, with Sir John Wolfe Barry, K.C.B., as Consulting Engineer. Mr. D. McLellan, M. Inst. C.E., acted as Resident Engineer during the earlier part of the work, and was succeeded by Mr. H. Cunningham, Assoc. M. Inst. C.E. Mr. Alexander Grant was the Superintending Inspector of Works.

The contractors were Messrs. Sir William Arrol and Company and Messrs. Morrison and Mason, Glasgow, and their engineering representatives on the works were respectively Mr. William Burnside and Mr. James Waddell, M. Inst. C.E.

The Paper is accompanied by twenty-two drawings and tracings, from which Plates 1 and 2 and the Figures in the text have been prepared; there are also the following Appendixes.

[Appendixes.

APPENDIXES.

APPENDIX I.

DEPTHS OF THE MAIN CAISSONS.

Pier.	Depth below High-water Level.	Depth below Surface of Quays.	Depth below Bed of River.
1. North land pier—	Feet.	Feet.	Feet.
Caisson A 1	40·30	50·10	
Caisson A 2	51·24	61·04	
2. North river pier—			
Caisson B 1	66·55	..	44·10
Caisson B 2	66·55	..	44·10
3. South river pier—			
Caisson C 1	70·47	..	48·02
Caisson C 2	66·43	..	43·98
4. South land pier—			
Caisson D 1	45·50	57·30	
Caisson D 2	45·40	57·20	

APPENDIX II.

DIMENSIONS AND WEIGHTS OF THE MAIN CAISSONS.

Pier.	Length.		Width.		Height from Cutting Edge to Top of Uppermost Skin-plate.		Weight of Steel.
1. North land pier—	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Tons.
Caisson A 1	89	6	20	3	41	1	195
Caisson A 2	61	8	20	3	52	1	167
2. North river pier—							
Caisson B 1	79	4	23	3	34	3½	203
Caisson B 2	52	3	23	3	34	3½	142
3. South river pier—							
Caisson C 1	73	10½	23	3	38	11½	195
Caisson C 2	48	7	23	3	35	0	133
4. South land pier—							
Caisson D 1	55	8	20	3	45	3	137
Caisson D 2	55	8	20	3	45	3	137

APPENDIX III.

WEIGHTS OF MASONRY AS ASCERTAINED BY EXPERIMENT.

	Lbs. per Cubic Foot.
Cement concrete (5-to-1)	130
Granolithic concrete (2-to-1)	138
Red brickwork in cement	122
Blue brickwork in cement	147
Freestone rubble masonry in cement	138
Freestone ashlar	145
Granite ashlar	164½

APPENDIX IV.

THE PNEUMATIC INSTALLATION—EXCERPT FROM CONTRACT SPECIFICATION.

"For the purpose of sinking the cylinders and caissons by the pneumatic process, such number of compressed-air installations shall be provided as will ensure safe and timeous completion of the works. The contractor shall, without extra charge beyond the contract prices, provide for each installation, in duplicate or in greater number as may be required, all engines, boilers, machinery, pumps, compressors, receivers, locks, high-pressure grouting appliances, hoisting gear and other apparatus, instruments, appliances and plant, together with all furnishings and fittings necessary to ensure the providing of an abundant and constant supply of compressed air of sufficient pressure. All the engines, boilers, machinery, appliances, plant and things for each installation shall be of such power as to maintain the necessary pressure, but the duplicate or other sets shall at all times be ready for immediate work, and be so connected that the stoppage of one set or of one boiler or engine or pump or appliance will have no prejudicial effect. The air-locks and working-chambers of the cylinders and caissons, as well as all other parts of the works, shall be properly lighted with electric light.

"The air-locks for the men, as well as those for the materials, shall be constructed of steel plates with strong doors. If necessary, the doors shall be interlocked, and the contractor shall in all cases, by means of the most improved arrangements and appliances, specially provide for the ready passage of the workmen through the locks and for their safety, and for the convenient introduction of plant and materials to the working-chambers, as well as for the removal and discharge of the excavations from the working-chambers. There shall also be provided numerous valves, gauges, taps, alarm whistles, electric or other signalling apparatus and speaking instruments for communication with the working-chambers, as well as means whereby passage of the workmen through the air-locks shall be gradual in respect of pressure and temperature. To ensure gradual change of pressure, the air taps in the man-locks shall be of such small bore as will prevent sudden change from the higher to the lower pressure or vice versa, and there shall be provided safety-locks and warm dressing-rooms for the workmen as well as a special compartment or hospital fully equipped for the treatment of any who may be injuriously affected by working under compressed air. The safety, comfort and convenience of the workmen shall be specially kept in view, and to regulate and ensure their continued good health, they shall be supplied with special clothing for the working-chambers and with warm jackets for wear after leaving work, and there shall be provided in the dressing-room hot water for washing and all conveniences for dressing and waiting as shall be directed. The men engaged in the working-chambers shall be on specially short shifts proportioned suitable to the working pressure, and it is recommended that, after leaving the working-chambers but before passing through the air-locks, they be supplied with suitable stimulant, and that arrangements also be made whereby they may, if necessary, have without delay the benefit of the advice of a special medical practitioner. Quick passage of the men through the air-lock shall be absolutely prohibited, and they shall be allowed such sufficient time there and in the dressing-room as will prevent subsequent injurious results.

"The compressed air shall be kept perfectly cool by cold water jacketing and otherwise, and special precautions shall be taken to ensure its purity in the working-chambers, the carbonic acid gas being kept low, and, as a test of the purity of the air, it shall be frequently analysed by a qualified chemist.

"The air-locks and other appliances shall be altered in position as the requirements necessitate, and sufficient air-pressure shall always be maintained night and day in the working-chambers, whether excavating and sinking is proceeding or not; and, in building the concrete hearting of the cylinders and caissons, as hereinafter specified, the air-pressure shall be constantly maintained until the work is entirely completed."

APPENDIX V—PROGRESS OF THE EXCAVATIONS IN THE CAISSONS.

(1) LAND PIERS.

Caisson.	Depth from Surface of Quays.	Character of Strata.	Depth of Excavations at Bottom on which Calculations are based.	Volume of Excavations.	Men-Hours.	Volume of Excavations per Man per Hour.	Time in Months from Commencement to Completion of Excavations in Working-Chamber.	Remarks.
	Ft. Ins.		Ft. Ins.	Cubic Yards.		Cubic Yards.		
A 1	18 0 37 0 50 0	Made ground . . . Muddy sand and gravel Sharp sand and coarse gravel	23 0	1,544	3,540	0·43	5·33	Depth of digging in working-chamber 43 feet; sand and gravel, but with some large stones; very coarse shingle.
A 2	22 0 56 0 61 0	Made ground . . . Muddy and stiff clay (average) . . . Sand and gravel (average) . . .	18 0	833	3,000	0·28	5·33	Depth of digging in working-chamber 51 feet; stiff clay, requiring special tools to cut.
D 1	11 0 22 0 40 0 48 0 51 6 57 0	Made ground . . . Muddy sand . . . Sharp sand . . . Gravel . . . . . Sharp sand . . . Sharp sand and gravel	24 4½	1,017	1,620	0·63	4·20	Depth of digging in working-chamber 50 feet; sand and gravel, easily handled.
D 2	11 6 24 0 44 0 57 0	Made ground . . . Muddy sand . . . Sharp sand . . . Sharp sand and gravel	24 9½	1,034	1,800	0·57	5·17	Depth of digging in working-chamber 50 feet; sand and gravel, easily handled.

APPENDIX V—*continued.*

(2) RIVER PIERS.

Caisson.	Depth from Bed of River.		Character of Strata.	Depth of Excavations at Bottom on which Calculations are based.		Volume of Excavations.	Men-Hours.	Volume of Excavations per Man per Hour.	Time in Months from Commencement to Completion of Excavations in Working-Chamber.	Remarks.
	Ft.	Ins.		Ft.	Ins.	Cubic Yards.		Cubic Yards.		
B 1	1	6	Mud	..		1,037	4,140	0·25	2·67	Depth of digging in working-chamber 38 feet; largely through clay, which was found difficult to cut.
	24	6	Muddy clay . . .	..						
	31	0	Clay with sand . .	3	4					
	43	0	Sand	12	0					
B 2	1	6	Mud	..		831	1,200	0·70	1·67	Depth of digging in working-chamber 38 feet; sand and gravel in bottom, and everything favourable.
	16	0	Muddy clay . . .	..						
	28	0	Sand with clay . .	3	2					
	43	6	Sand and gravel . .	15	6					
C 1	1	0	Mud	..		1,216	2,280	0·55	3·75	Depth of digging in working-chamber 42 feet; sand and gravel, easily handled.
	11	0	Muddy sand . . .	..						
	34	9	Coarse sand . . .	6	1					
	40	0	Gravel	5	3					
	43	0	Sand	3	0					
	48	0	Sand and gravel . .	5	0					
C 2	1	0	Mud	..		819	800	1·02	2·75	Depth of digging in working-chamber 40 feet; sand and gravel; easily handled; men on bonus system.
	12	0	Muddy sand . . .	..						
	34	9	Sand	10	10					
	39	6	Coarse sand . . .	4	9					
	43	9	Sand and gravel . .	4	3					

(*Paper No. 3824.*)

"The Queen Alexandra Bridge over the River Wear, Sunderland."

By FRANCIS CHARLES BUSCARLET, Assoc. M. Inst. C.E., and
ADAM HUNTER, M. Inst. C.E.

THE bridge and approaches which are the subject of this Paper convey both a road and a double line of railway across the River Wear, at a point about 2½ miles from the entrance to Sunderland harbour.

Previous to 1899 the necessity for a road-bridge over the river in the neighbourhood of the new bridge was realized, and rough plans and estimates were prepared, but it was not until 1899 that the North Eastern Railway Company finally came to an agreement with the Corporation of Sunderland. It was then decided to construct a joint bridge across the river on the present site, and parliamentary powers for the undertaking were obtained in 1900.

The general design of the main bridge will be readily understood from Plate 3. It consists of three 200-foot (clear) land spans, one on the south and two on the north side of the river, and of a 330-foot (clear) span over the river, this last giving a clear headway of 85 feet above high-water level of spring-tides. The massive abutments and piers which support these spans are built of Norwegian rock-faced granite in cement.

SITE AND APPROACHES.

The railway (Fig. 1, Plate 3) extends from a point (A), about 160 yards west of Millfield station on the Penshaw branch railway, to a point (B), opposite to the Hylton colliery on the Hylton, Southwick and Monkwearmouth branch railway; a total distance of about 1 mile 59 chains. It is built primarily to enable the large mineral-traffic from the coal-fields, in the district between Washington and Annfield Plain, to be brought, if required, to the South Dock, Sunderland, without the two reversals that are now necessary at Washington and Penshaw. The aggregate output from these collieries is upwards of 6 million tons per annum. Ultimately, the new bridge and railway will be used to develop passenger-traffic in the district.

The roadway extends below the railway from Havannah Street (C) on the south side of the river to the point D on the north side, where it leaves the railway and, crossing Camden Street, leads up to Mary Street bridge; this being the proposed route for the tramways that are eventually to be laid across the new bridge. The building of the roadway from point D to Mary Street bridge, and the widening of this bridge to suit future requirements, was carried out by the Southwick District Council.

When laid, the tramway will connect the system which has its terminus at "The Green," Southwick, on the north side of the river, with that which, on the south side, extends westwards from the Central station, Sunderland, along the Hylton Road and past Millfield station. The new road-bridge will also be of great value for foot-passengers between the industrial districts of Sunderland and Southwick, and for the heavy traction and other traffic between Sunderland and Newcastle, as the only other means of communication between the banks of the river in the immediate neighbourhood is a road-bridge crossing the river at a point about 1 mile 3 furlongs farther down. Lastly, the new bridge is made use of to carry important water- and gas-mains across the river.

Approach-Railways.—Starting from the south side of the river at its junction (A) with the Penshaw branch, the railway (Figs. 1 to 3, Plate 3) is laid on an embankment in which are two small girder-bridges, one over a proposed road and the other over the Lambton railway (a colliery-line). From this point it is carried on retaining-walls as far as a girder-bridge over New Road, and beyond this again on brick arching, until a girder-bridge over Havannah Street is reached at which point (C) the south roadway-approach to the main bridge commences.

From point (E) on the north side of the river the railway is carried on brick arching as far as two girder-bridges over Wear Street and Camden Street, then on an embankment as far as the skew girder-bridge over the Hylton, Southwick and Monkwearmouth branch railway, and the adjoining road; and lastly, beyond that again on an embankment which tails out at (B), opposite to the Hylton colliery.

Owing to the high level of the railway, a great deal of the masonry in connection with this part of the work is of massive character; it is built of local stone, faced with red sandstone from the Locherbriggs quarry, near Dumfries. It was possible to get blocks of stone from this quarry of almost any size; its fine colour and the fact that it is easily worked, though hardening considerably on exposure, render the stone valuable for building purposes, from both an architectural and an engineering point of view. A block of this stone, 6 inches cube, was submitted to a crushing-test and withstood a weight of 90 tons (i.e., 360 tons per square foot), the edges only being slightly split.

The embankments are made up mostly of material from the spoil-bank at the Wearmouth colliery.

There is nothing of particular interest from an engineering point of view in connection with this part of the railway, except the skew girder-bridge referred to. This bridge is built at the exceptional angle of 18°, the skew and square clear spans being respectively 212 feet and 65 feet. The steelwork of the superstructure weighs 743 tons.

South Approach.—At Havannah Street bridge begins the south approach-road, which is 346 feet long and rises towards the main bridge at a gradient of about 1 in 44. It consists of a short length of retaining-walls and several brick arches, the latter being built on solid ashlar piers which taper to a width of 5 feet below the springers (Figs. 3 and 4, Plate 3). The piers, arch-quoins, parapet-walls and facework are built of the red sandstone previously referred to.

Owing to the depth of made ground it was found necessary to carry the foundations below the level originally intended, and 6-to-1 cement concrete was used where necessary below the ashlar footings, all the foundations being taken down to good clay. Inside the retaining-walls, and behind the 3 feet of masonry above the arching, a backing of 6-to-1 cement concrete, respectively 10 feet and 11 feet 3 inches wide, is built up; this also forms a solid foundation for the granite blocks that act as bases for the steel columns supporting the railway superstructure. These steel columns are placed 12 feet 6 inches apart longitudinally and are made up of two channels with lattice bracing. They are anchored to the bedstones by bolts and washer-plates, and are secured longitudinally to each other by lattice girders 4 feet high at the level of the footways. These girders act also as a handrailing between the roadway and footways. The bases of the columns are strengthened with gusset-plates and angles, and are surrounded with bitumen concrete composed of pure pitch and oil, and coke breeze. All the steelwork with which the bitumen concrete was to come in contact was carefully dried before the concreting was allowed to proceed, and the coke breeze was thoroughly heated in a special drier. It was then laid on the steelwork in 2-inch to 3-inch layers and thoroughly grouted up with a mixture of hot pitch and oil. This bitumen concrete upon a ¾-inch layer of rock asphalt was also used instead of cement concrete on the roadway and footways over the main bridge; it was considered to afford a better protection against damp, and was also considerably lighter than cement concrete, the respective weights being 89 lbs. and 140 lbs. per cubic foot. This modification reduced the weight of the superstructure over the river and land spans by 104 tons and 67 tons respectively, or by 6 cwt. per lineal foot.

The superstructure which carries the railway is arranged to give a clear headway of 18 feet above the finished road (Figs. 4, Plate 3). It consists of cross girders riveted to the columns by means of brackets, with ⅜-inch floor-plates in between; there are longitudinal rail-bearers under each rail, and rail-guards and angle cleats are provided for the permanent way. The lattice-work parapets are 6 feet high, being placed 28 feet apart between centres. The columns at their tops are braced together longitudinally with lattice-work. They are also braced together transversely where possible above the headway, and longitudinally at the ends and at two intermediate bays, by a system of channels and tee-bars to stiffen the superstructure. Provision is made for expansion over these two intermediate bays.

The total weight of steel in the superstructure of the south approach is 453 tons.

The drainage of the railway superstructure is led into cast-iron gutters, which run below the rail-bearers and alongside every alternate cross girder; they empty into down-comers which deliver into gullies on each side of the roadway.

The roadway has a camber of 3 inches and consists of beech-wood blocks laid on 8 inches of cement concrete with a cushion of sand. The footways, which are formed of 3-inch cement concrete on broken bricks, have a fall of 2 inches towards the roadway.

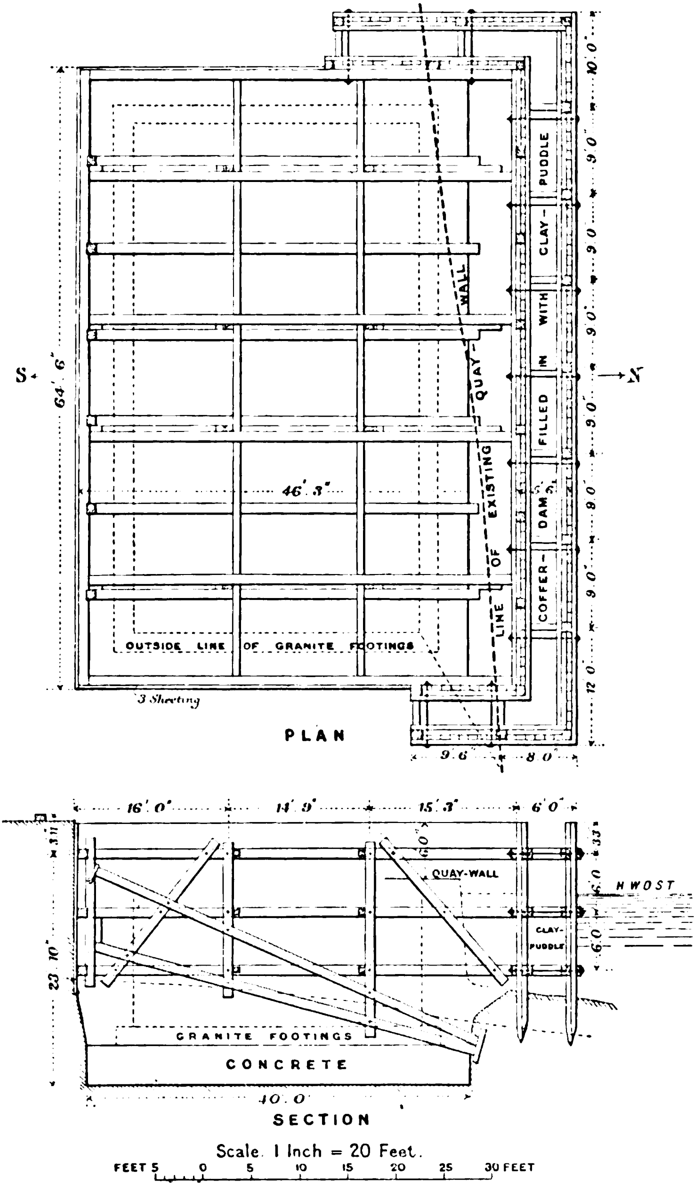
North Approach.—On the north side of the river the railway turns out (Fig. 2, Plate 3) in a westerly direction with a 19½-chain curve; while the roadway branches off in the opposite direction, at an angle of about 8° with the centre-line of the bridge. The construction of this approach is similar to that on the south side, and the total weight of steel in the superstructure is 280 tons.

FOUNDATIONS OF RIVER PIERS.

The north and south abutments, and pier No. 3 on the north side of the river, are founded on good stiff clay, with 6 feet to 7 feet of concrete below the granite footings.

The excavation of the foundations for pier No. 1 necessitated the building of a coffer-dam; but as the river side of the pier was approximately on the line of the existing quay-wall, it was only necessary to build the coffer-dam on that side, with short return ends carried back to the solid ground (*Figs. 5*, p. 64). The coffer-dam was of special construction as it was built upon a rock bottom. The bed of the river on the south side being limestone under a foot or two of mud, the piles had practically no foothold, and the dam had to be kept in position by connection to the struts inside and by means of wire ropes placed outside and secured to the ground behind. The other three sides of the foundations were supported by 3-inch sheeting, that on the land side being kept up by long raking struts which were carried down to the rock inside the dam on the river side. The placing of these struts was an important matter, as before they could be fixed permanently a certain amount of excavation had to be done, and until the struts were in position nothing was secure. The risks were that the dam would be pushed out into the river either from the fall of the tide or from a rush of water, which threatened to come over the sides of the coffer-dam when ships were being launched from the adjoining yards. The foundations were carried down to 20 feet below high-water level, a perfectly sound and almost level bed of limestone being reached at this depth. The limestone was first met with at about 8 feet below high-water level; its surface, which consisted of soft rock very much fissured, was loosened by means of picks and bars and, as the rock became harder, by means of blasting, while the last foot or two was quarried out so as not to disturb the underlying strata. The rock at the bottom of the foundations was left with an uneven surface, so as to form a bond with the concrete above. Many of the blocks of stone quarried out, being of large size and having level beds, were used as backing in the masonry retaining-walls. Before the quarrying-out was stopped, bore-holes were driven another 16 feet into the rock by means of steam-drills, thus ensuring that the foundations were perfectly sound.

Figs. 5.


S
N
64' 6"
46' 3"
10' 0"
9' 0"
9' 0"
9' 0"
9' 0"
9' 0"
9' 0"
12' 0"
COFFER-DAM FILLED IN WITH CLAY-PUDDLE
LINE OF EXISTING QUAY-WALL
OUTSIDE LINE OF GRANITE FOOTINGS
3 Sheeting
PLAN
9' 6"
8' 0"
16' 0"
14' 9"
15' 3"
6' 0"
3' 11"
23' 10"
6' 0"
QUAY-WALL
3' 3"
6' 0"
H W O S T
6' 0"
CLAY-PUDDLE
GRANITE FOOTINGS
CONCRETE
40' 0"
SECTION
Scale. 1 Inch = 20 Feet.
FEET 5 0 5 10 15 20 25 30 FEET

TEMPORARY COFFER-DAM FOR THE FOUNDATIONS OF THE SOUTH RIVER PIER.

An average of 4½ feet of concrete was then laid over the whole area, and on this the granite masonry was built. The struts and rakers had to be removed in building up the granite, and the sides were supported from the masonry. These temporary struts and rakers were shifted up as each course was completed and the course above became sufficiently advanced to support them.

Although limestone was found on the south side of the river at 8 feet below high-water level, this bed extended only about half-way across the river, and practically no stone was met with on the north side until a depth of 75 feet had been reached. The foundations of pier No. 2 on the north side of the river, therefore, were carried down to the red sandstone, or 78 feet 6 inches below high-water level.

The excavation of the foundations for pier No. 2 necessitated the sinking of a rectangular caisson. It was at first intended to use a temporary caisson above the permanent shoe, inside of which the masonry was to be built up in the usual way; but at the request of the contractor and on conditions laid down by the Chief Engineer, the temporary caisson was dispensed with, and the sinking was carried out by superimposed load. The conditions under which this was allowed were, that no payment was to be made until the caisson had been satisfactorily sunk on to a good foundation and the superstructure of granite had been brought up to high-water level.

The caisson (Figs. 6, Plate 3), as constructed for these altered conditions, measured 63 feet 3 inches by 35 feet 3 inches over the cutting edge; it was 44 feet deep and weighed about 340 tons.

The shoe (Figs. 7, Plate 3) consisted of ⅞-inch shell-plates, to the inside of which were riveted triangular brackets. The ½-inch plates which formed the roof and sloping sides of the working-chamber were attached to the top and sides of the brackets. Manholes and smaller holes for concreting were provided in the roof-plate, over each pocket formed by the brackets.

The cutting edge of the shoe was formed by strengthening the shell-plates on the outside by two ¾-inch plates, respectively 22 inches and 14 inches deep; and on the inside by a 1-inch plate bent to suit the base of the shoe-brackets, with a 6-inch by 6-inch by ⅞-inch angle above it. Cover-plates and angles are provided at the shell-plate joints.

Above the shoe, the shell-plates are ⅝ inch thick, and are connected to cross girders, which rest on the brackets at the two sides of the caisson. These cross girders are strengthened and braced together by angle stiffeners and diaphragms.

Above the cross girders, the caisson is built up of five strakes of ½-inch shell-plates and one top strake of ⅜-inch shell-plates, the longitudinal joints being 3 inches deep. Each strake overlaps the one above, and the caisson is thus reduced in length and breadth at every strake, which gives the required taper. The caisson is stiffened by angles and cross girders as shown in Figs. 6. Two openings for 38-inch shafts were allowed for in the roof of the working-chamber, but eventually only one at the east end was used.

The first three strakes of shell-plate were built up on timber staging 12 feet above the river-bed, that is, about 4 feet above high water. After grouting up the bottom of the pockets above the cutting edge with 6 inches of 1-to-1 Portland cement and sand, the caisson was lowered, by four hydraulic jacks, on to the bed of the river, which had been levelled over the required area.

The pockets and the space between the girders forming the sides and roof of the working-chamber were then filled in with concrete, commencing at the north side of the caisson, which was the higher. Special care was taken to see that this was properly done. At first the concrete was brought up level to within 6 inches or so of the deck-plates, and after it had been well rammed, the remaining space was packed with fine concrete by means of grafting-tools and iron bars. This was found to be unsatisfactory, as the concrete got jammed in under the deck-plates, and prevented the pockets from being thoroughly filled. The method then adopted was to bring the concrete in the pockets to within a few inches of the top, and to pack in through the manholes until it rose to the top of the two smaller holes. In addition to this, the corner pockets, which were more difficult to pack, were grouted up with cement.

From this point the sinking was carried out in the usual way by superimposed load, which consisted of the concrete filling of the caisson together with the granite. The top of the caisson, and afterwards the courses of granite, were kept sufficiently far ahead of the sinking to ensure that the top of the building would always be a few feet above high-water level; and to do this the concreting had often to be carried on day and night. Particulars of the sinking of the caisson are given in Appendix I.

Excavation was carried on without interruption during the sinking, except at the change of shifts or when the air-pressure had to be cut off. The method adopted throughout was to excavate from the centre towards the ends and sides, leaving a dumpling sloping up to the roof of the working-chamber either round the sides, or at both ends, or at one end only, according to the lie of the strata and the level of the caisson at the time. By this means the caisson was kept so fully under control that it could be brought almost to a level position, if required, and could be so guided as to be kept exactly on the centre-line.

During the sinking of the caisson various strata were met with. After passing through 19 feet of mud and yellowish sand, and 12 feet of sandy clay, the north and west cutting edges came on to stiff grey clay which fell in a south-easterly direction and at a slope of about 1 in 5 for two-thirds of the length of the caisson; it then dipped down at a very steep slope of approximately 2 to 1. The south side and east end of the caisson were thus on sand, while the north side and west end were on clay. As the caisson sank far more easily in the latter, this side and end had to be kept well banked up until the composite strata, which extended to a depth of about 23 feet, had been pierced; so much was this the case that at one time the bank extended from the west end to beyond the line of the opening into the unused shaft. Had that shaft been kept open, as was originally intended for hurrying forward the excavation, it could seldom have been used during this period of the sinking. The caisson next passed through 5 feet of clay extending over the whole area of the cutting edge, and from that time the tendency was for it to travel faster at the east end, consequently that end had to be banked up. After the clay came yellow sand with lumps of half-formed sandstone, extending a few feet inside the north cutting edge, and with gravel over the remaining area. This continued for a depth of about 6 feet, the area of yellow sand gradually increasing, and that of the gravel decreasing, until grey seggar (fire-clay) was met with. When about 3 feet 6 inches of this seggar had been passed through, a stratum of 12 inches of hard grey sandstone was encountered. This was at a depth of about 76 feet below high-water level, and it was decided to take off the air-pressure gradually, to see how far the caisson would travel. The superimposed weight was then 9,890 tons (neglecting friction). This was done, and no movement was noticed until the pressure had been reduced to 12 lbs. per square inch, when the caisson began to descend; and when the pressure had been reduced to zero it had sunk 7 inches. On restoring the air-pressure it was found that the strata of grey sandstone had broken and tipped up under the weight of the caisson, leaving the cutting edge resting on hard, red, laminated sandstone. About 18 inches of this sandstone was passed through, and the cutting edge of the caisson was then 78 feet below high-water level.

Before deciding to stop at this depth, trial holes were dug to satisfy the resident engineers as to the condition of the underlying strata. This being satisfactory, the air-pressure, which at this point reached a maximum of 39 lbs. per square inch, was taken off gradually, causing the caisson to sink a further 2 inches into the sandstone. With regard to level, taking 0 as the level of the highest corner, the caisson at this stage stood as follows: north-east corner, 0; south-east corner, ¼ inch down; north-west corner, 1 inch down; and south-west corner, ¼ inch down.

The air-pressure was again restored, and the concrete was filled into the working-chamber, commencing at the west end and working out towards the east shaft, until it extended to within a few inches of the roof. The remaining space was then packed as tightly as possible by ramming in the concrete. When this had been done, neat cement grout was poured into the working-chamber through a 3-inch pipe, which extended from the air-lock, down the east shaft and across the caisson. The pipe was suspended from the roof of the working-chamber as far as the unused shaft at the west end, and rose a few inches up it. The grout was poured down this pipe until it rose in the shaft 2 inches above the roof of the working-chamber, and when it remained at this level the grouting was stopped. Altogether about 1 ton of cement was used. When the cutting edge had been sealed, an escape-valve was provided on the shaft and the compressor was run slowly, so as to prevent the air-pressure from rising any higher than 33 lbs. per square inch, which was the pressure used during this final grouting. When the grouting had been completed the air-pressure was finally taken off, and the air-lock and shafts were removed, the holes occupied by the latter being filled with concrete.

The total weight on the foundations at the close of these operations was 9,890 tons (neglecting friction). During this concreting the caisson continued to descend for a few days, sinking a total of 6 inches, when it stopped at 78 feet 6 inches below high-water level of spring-tides. The working-chamber was then a little over one-third full of concrete, and the levels were as follows: north-east corner, ¾ inch down; south-east corner, ⅞ inch down; north-west corner, 0; and south-west corner, 0. No subsequent change of level, even after the pier was built to its full height, was observed.

All the granite was built in 3-to-1 cement mortar, and great care was taken with the grouting of the joints. The outside joints of each course were first plastered with cement mortar, and the inside joints were then well filled with a very liquid cement grout that was swept into them until they were full, after which the grout was allowed to settle. This was done repeatedly until the joints were quite full, and no further settlement being detected, the next course of granite was commenced. The granite masonry in the piers, apart from the relieving arches, is solid throughout, while the abutments have a central and two side pockets measuring 16 feet by 12 feet and 8 feet by 7½ feet respectively.

The foundations, embankments and masonry were commenced in February, 1905, and were practically completed by July, 1907; the amount of material used was as follows:—

Granite	496,000	cubic	feet.
Ashlar	222,000	,,	,,
Rubble	19,000	,,	yards.
Concrete	13,000	,,	,,
Brickwork	6,000	,,	,,
Earthwork	330,000	,,	,,

This work, with the exception of the sinking of the caisson by the contractors, Sir William Arrol and Company, was carried out by the sub-contractors, Messrs. Mitchell Brothers, of Glasgow. Taking into consideration the fact that there was unavoidable delay at the beginning, owing to a large portion of the site for the work not being available, and later by the stopping of the supply of granite from Norway owing to the frost, the progress was distinctly satisfactory.

SUPERSTRUCTURE.

Land Spans.—Each of the three land spans (Fig. 3, Plate 3) consists of two parallel main girders of the Linville type, 224 feet long and 30 feet deep, placed 32 feet apart from centre to centre. The roadway floor is carried from the bottom booms, and the double-line railway-track is placed between the main girders above the roadway, allowing 18 feet of clear headroom. The main girders are sufficiently braced together by the railway cross girders, and do not require any special bracing-girders or wind-bracing. The general construction is similar to that of the river span described later.

The total weight of steelwork in each land span is about 1,000 tons, and the total dead weight including permanent way is about 1,400 tons.

River Span.—The main or river span (Figs. 10, Plate 4) consists of two curved-top girders of the Linville type, 353¾ feet long over end-plates; they are 30 feet deep over angles at the ends, rising to 42 feet over angles at the centre, and are placed 32 feet apart from centre to centre. The booms are of trough section and are composed of web-plates (two side and one centre) and flange-plates, connected together by means of angles and stiffened by vertical diaphragm-plates placed at the centre of each bay. The side plates are 42 inches deep, and the flange-plates are 68 inches wide in the top and 56 inches wide in the bottom booms.

The end posts, which measure 9 feet by 4 feet 2 inches over plates, are built up with a centre plate and four outside plates, connected together by angles and stiffened by three horizontal diaphragms.

The vertical posts are 12 feet apart between centres, and consist of a web-plate and eight channels. They are designed to saddle over the centre web-plate of the bottom boom, and to receive that of the top boom.

The diagonal ties, with the exception of the four end ones which extend across the posts and one bay, extend over two bays and consist of flat bars in pairs; and towards the centre, where the stresses are liable to be reversed owing to the moving load, they are braced together with lattice bars. They continue beyond the centre as counter-ties; and are connected by gusset-plates to both booms.

The top booms are connected by six overhead bracing-girders at 24 feet intervals, these being used only towards the centre of the span, where the headroom for the trains permits of them.

The main girders rest at one end on roller bearings, and at the other end on fixed bearings (Figs. 9, Plate 3); provision for expansion is thus made at one end of the span.

The roadway is carried on cross girders which are suspended from the bottom booms of the main girders by suspender angles (Figs. 10, Plate 4). These are spaced 3 feet on each side of the vertical posts and 6 feet between centres, and they project as cantilevers about 13 feet 6 inches beyond the main girders on each side to carry the footways, and, outside of these, the gas- and water-mains.

These cross girders are 64 feet long over all, and are 2 feet 9 inches deep over angles between the main girders. Longitudinal stringers 5 feet apart are riveted to the webs of the cross girders, and inverted buckled plates $\frac{3}{8}$ inch thick are riveted to the cross girders and stringers to form a decking for the roadway. Instead of being in separate plates, the decking was made in lengths with several buckles in each length. On the projecting cantilevers, longitudinal troughing is laid to carry the footways. Outside the footways, which have a lattice-work parapet 4 feet 6 inches high, steel chocks are fixed to support the gas- and water-mains. A bulb-tee is placed at the extreme end of the cantilevers to carry a permanent travelling gantry, which is hinged to pass the piers. The roadway has a clear width of 26 feet, and is formed of 5-inch beechwood blocks laid diagonally, with a $\frac{1}{2}$-inch cushion of sand, upon bitumen concrete, this being filled into the buckled plates and above them. The concrete varies in thickness from 8 inches at the centre to 5 inches at the sides of the road, thus giving the required 3-inch camber. Before laying the bitumen concrete the buckled plates were covered with ¾ inch of rock asphalt, which was increased to 1 inch over the top of the cross girders and stringers to protect the rivet-heads; it was carried 12 inches up the main girder web-plate, to form a fillet on either side of the roadway.

Cast-iron troughing lined with rock asphalt is laid on each side of the roadway for drainage purposes, the space between it and the asphalt filleting being filled with bitumen concrete.

In the centre of the roadway 12-inch by 6-inch waybeams are laid over the two centre stringers to carry a single line of tramway, and are held in position by angle cleats riveted to each cross girder.

The inside of the lower booms is lined at the bottom with rock asphalt, which averages ¾ inch in thickness and slopes towards 1½-inch holes provided in each bay for drainage.

The footways, which are 7 feet wide and have a slope of 2 inches towards the parapets, are formed of a 2-inch layer of 4-to-1 gravel and cement, with a 1-inch layer of 2-to-1 granite chips and cement on top. This cement concrete is laid upon bitumen concrete, which was filled into and over the troughs after these had been covered with ¾ inch of rock asphalt. There was some difficulty at first in getting the rock asphalt, which was tried of various consistencies, to adhere to the steelwork, even when the latter was perfectly clean; but a satisfactory result was ultimately obtained by covering the cleaned steelwork with a thin coating of creosote oil, well rubbed in and then left to dry thoroughly before the rock asphalt was laid over it.

The railway (Figs. 10, cross section) is carried on cross girders 12 feet apart from centre to centre, and resting on brackets; both cross girders and brackets being riveted to each other and to the end posts and verticals.

The cross girders are 2 feet 9 inches deep over angles, and they are arranged at a similar height to those in the land spans. Longitudinal rail-bearers are riveted to the webs of the cross girders. Flat plates ⅜ inch thick are riveted to the cross girders and rail-bearers and form the floor of the railway, being covered with $\frac{3}{16}$ inch of "Mastico" bituminous enamel.

The permanent way is similar over the whole bridge and south approach, and is composed of 95-lb. British Standard steel rails with specially-rolled continuous fishplates (Figs. 11, Plate 4); these rest on kyanized timber waybeams, which are scarfed and bolted together every 36 feet or so. Brackets support the rail-guards, which consist of 8-inch by 3½-inch channel-bars placed so that their top is 2 inches above rail-level. As the rails do not rest on chairs, the necessary cant is given to them by the waybeams, which are sawn to the required slope. Provision for expansion is made in the continuous fishplates as shown in the Figure. The north approach, being on a curve, the continuous fishplates could not well be used, and the permanent way here consists of steel rails with ordinary chairs and fishplates.

The total weight of steelwork in the river span is 2,600 tons, or 7½ tons per lineal foot, and each main girder weighs 954 tons. The total dead weight, including permanent way, etc., is 3,200 tons, which is equal to 9 tons per lineal foot. The weight of the river span is not exceeded by any existing bridge of the same length.

The steelwork in the bridge and approaches was designed to carry the following live loads: 1½ ton per lineal foot on each railway track; an equally distributed load of 1 cwt. per square foot on the whole area of the roadway and 100 lbs. per square foot on the footways, or a concentrated load of 40 tons on four wheels on the roadway floor-girders.

The maximum working-stress allowed was 6½ tons per square inch on the net section.

The riveting throughout the whole bridge was done by means of hydraulic and pneumatic long-stroke riveters. The hydraulic riveters were of two types, hinged and fixed, the former being used for the flange- and the latter for the web-plates of the booms. The rivets were usually heated in the ordinary way, but at times, especially for the hydraulic riveters, which require a more uniform supply of rivets, a heating-furnace with an electric blower was used.

Manufacture.—All the steelwork in the contract is of mild steel, specified to have an ultimate tensile strength of not less than 27 tons and not more than 31 tons per square inch, with an elongation equal to at least 16 per cent. on a length of 8 inches before fracture.

The girders were drawn down full size on the mould-loft floor, from which templates of well-seasoned yellow pine were made. These templates were applied to the straightened plates and bars, which were marked off to enable them to be planed to shape and drilled. After the plates and bars were planed to shape, the component parts of the flange or webs of a boom were assembled, clamped together and marked from the templates; all holes were drilled their full size through the total thickness of metal, thus ensuring that they would come together correctly after the parts had been taken apart and cleaned.

The three land spans, being exactly similar, were marked off and drilled from the same templates. One span only (the last one required at the site) was erected in the yard, to "prove" the correctness of the manufacture, the remaining two spans being sent direct to the site. As the river span is symmetrical, only one half was erected in the yard previous to inspection and dispatch.

Before leaving the manufacturer's yard the steelwork received a coat of boiled oil, and when erected and properly scraped and cleaned, one coat of red lead and two of best "Torbay" paint; excepting the underside of the spans, the ends of the main girders, and the inside of the top and bottom booms, which were painted with two coats of Brigg's solution. The railway deck of the main span and approaches, and of all girder-bridges not to be ballasted, were covered with $\frac{3}{16}$ inch of "Mastico" bituminous enamel.

ERECTION OF THE LAND SPANS.

The land spans were erected in their permanent position upon staging formed of three timber trestles; these supported temporary girders to form a platform about 4 feet below the bottom booms of the main girders. The highest trestle was 72 feet from ground to deck level; it was formed of two piers, 11 feet square, which were placed 32 feet apart between centres, and braced together and strutted on the outsides to give a transverse width of 64 feet at the base. Each pier was formed of four vertical timbers, 12 inches square and braced together on the four sides; excepting, however, at the front of the pier next the 65½-foot span, where two timbers were used under each longitudinal staging girder, the load being limited to a maximum of 24 tons on each vertical timber. In the three smaller spans of the staging the permanent roadway cross girders of the river span were used as temporary staging girders. The staging was taken down and re-erected for the two remaining land spans.

The cross girders of the roadway were first laid down on the platform and wedged up to suit the building-camber. When this had been done the bottom flange-plates were laid on them, and the centre and side webs were placed in position. The end posts and the verticals were next erected, and simultaneously with the latter the diagonal ties and the top booms.

The main girders were erected with the building-camber of 4 inches at the centre of the span, this falling to about 2½ inches under the full dead load.

The erection of the three land spans was commenced in April, 1907, and practically completed by March, 1908, those adjoining the river span being finished first.

The quantity of temporary work in the staging, per ton of permanent steelwork erected, amounted to, 10 cubic feet of timber in the trestles and platforms, 5 cubic feet of timber in the blocking, gangways, riveting-platforms and painting-stages, and 1·68 cwt. of steelwork in the staging-girders, which includes the permanent cross girders used in the two small spans. As this staging was taken down and re-erected for the other land spans, the net quantities are one-third of these amounts and compare very favourably with the amounts used in the erection of large spans of other bridges.

ERECTION OF THE RIVER SPAN.

The shipbuilding yards on both sides of the river extend close up to the bridge, and new vessels are launched directly under it. This condition, as well as the necessity for keeping the navigable waterway always open for the passage of vessels, led to the insertion in the North Eastern Railway Act of 1900 of a clause for the protection of the River Wear Commissioners. The clause required that the bridge should be constructed with only one arch or span over the river, of not less than 321 feet on the square, between the bases of the piers, and with a headway of not less than 85 feet above high-water level of spring-tides. Further, during the time of erection, the Company should keep the navigation of the river, at and about the bridge, free and clear, subject, however, to provisions allowing coffer-dams, piling, etc., to be temporarily placed where necessary for the erection of the bridge-piers. Upon this understanding, the Act was passed authorizing the construction of the bridge.

The contract specification, framed on these requirements, suggested two alternative schemes by which the river span might be built, namely:—

(1) By using temporary towers and suspension members attached to the bottom booms and anchored to the land spans.

(2) By using the land-span girders cantilevered over the river, strutted out from the piers and anchored firmly to the shore, supporting a platform upon which the main girders might be erected.

The contractor was not bound to any of these methods of erection, if he could propose any other scheme which should be approved by the Chief Engineer.

Before the contractor decided on the method of erection, comparative estimates were made of the cost of different schemes. Building the main girders back on the land spans and rolling them forward was considered, as well as building them upon brackets or fan-like staging supported upon the river piers; consideration was also given to a scheme for erecting across the opening temporary girders to which the permanent roadway cross girders would be secured, forming a platform upon which to build the main girders. The great weight to be dealt with rendered all these schemes very costly, so that it was finally decided to convert the main girders temporarily into cantilevers anchored back to the adjacent land spans, and to build them by overhang from each side of the river. From the weights previously given it will be seen that this is the heaviest independent span yet erected by overhang.

The general scheme of the erection of the river span is shown in Figs. 13 and 14, Plate 4. The adjacent land spans had been entirely completed previously, and the temporary gussets for the lower end of the back anchor-ties had been riveted up to the side plates at the far end of the bottom booms.

A 10-ton steam-crane was placed on the railway floor on each side of the river, and moved forward as the erection proceeded.

Stage 1.—After setting the bearings for the river span upon the piers the first length of the bottom boom was placed in position, and temporarily held back to the end posts of the land span while the end posts and first diagonal ties of the river span were erected and riveted. The temporary tie used for this is shown dotted in Fig. 13. The girder was now adjusted to the correct alignment, and to a suitable inclination to allow for deflection during building, by means of the long temporary bolts at the top of the end posts. Solid steel thrust-blocks were placed between the end posts of the two spans at the level of the bottom booms, to transmit the thrust from the river span to the bottom booms of the land spans. These thrust-blocks were 60 inches wide by 45 inches deep, and were made up of four thicknesses of machined slabs having a total thickness of about 12 inches. When the booms and end posts were completely riveted the ends of the bottom booms of both land and river spans were carefully chipped and surfaced, using the abutting slabs as surface plates, so as to obtain an even bearing over the whole section. Three of the slabs were fitted into position, leaving a gap of about 1¼ inch, to which a slight taper of about ⅛ inch in plan was given. The fourth slab which was machined to gauge and made in three horizontal sections of the full width, was forced into the gap by hydraulic power. These closing sections were grooved to contain a heavy lubricant of plumbago and oil, in order to permit of easy vertical motion during the future operations of raising the projecting ends of the girders.

The expansion-bearings under the river span were then locked to prevent any motion, and the bolts were removed from the upper castings of the fixed bearings at the far end of the land spans, thus allowing the main girders to slide on the top of the cast-steel bearings, which had been prepared with oil-grooves and lubricated. Provision was also made at this end of the land spans to prevent any lateral motion.

The first vertical post and a length of the top boom were now put into position, and a temporary flange-tie and side web-ties at the tops of the land-span end posts were connected to the top flange and web-plates of the river span (Fig. 13, Plate 4). The temporary side ties were each formed of two short plates, 24 inches wide by ⅞ inch in thickness and of varying lengths, riveted to the web-plates on each side of the main girders. The horizontal flange-tie, shown by the thickened line in Fig. 13, was 63 inches wide by ¾ inch in thickness. The side web-ties were of sufficient strength to carry the whole load due to the overhanging weight, but it was considered that their short length would prevent them from accommodating the small variations due to the necessary lateral adjustments of the main girders during building. These adjustments would, probably, have thrown a greater tensile stress on the temporary ties on one side of the main girders than could safely be permitted, and the horizontal flange tie therefore was added as a precaution against this condition. To provide for an even distribution of the shearing stress over the rivets connecting the temporary ties to the main girders, the following procedure was adopted: The steelwork was erected up to the first joint in both booms and included the vertical posts, ties, and cross girders, as shown in Fig. 13. This was supported by the timber blocking at the front of the masonry pier, and by the tie-bolts near the top of the end posts. At this stage one-half of the holes (shown blackened) connecting the temporary horizontal side ties to each main girder were drilled and riveted, and the temporary tie-bolts and timber packings were removed. An additional length of the booms, with the posts, ties and cross girders, was added, so that the average shearing stress on these rivets from the overhanging weight was about 1¼ ton per square inch. The remainder of the connecting holes in the temporary side ties were now drilled and riveted as well as those in the horizontal flange-ties. The span was then built out to complete the second stage (Figs. 14, Plate 4).

Stage 2.—During the progress of building this stage, temporary steel towers 70 feet high were erected upon the end posts of the river span, being bolted down to the main girders. The holes in the bases of the towers had been drilled to a template taken from the girders, and after placing the lower length of the towers in position these holes were rimered. All the joints in the towers and the attachment of the large gussets at the tops were connected by turned and fitted steel bolts in holes rimered after erection, the bolts themselves being turned to gauge to an absolute driving fit. Each tower was formed of two plate-girder sections 54 inches deep, placed transversely across the booms and braced together in elevation as shown in Figs. 8, Plate 3. The towers were 9 feet wide at the base, tapering to 3½ feet at the top, and each had a sectional area of 190 square inches. Suitable diagonal bracing was fixed between the towers, to transmit the wind-stresses to the piers. In view of the heavy load that would come on to each tower, about 760 tons, it was considered desirable to distribute this weight evenly upon the whole section of the end post, without placing any reliance upon side riveting. To attain this end the bottom of each tower and the top ends of the plates and angles of the end posts were chipped flush, after the vertical side plates had been riveted to them.

The anchor-ties were erected from the top of the towers towards the lower ends, being supported on timber trestles placed about 30 feet apart. The lower end of each tie next the gusset on the land spans was left as a closing joint to be drilled at the site. Before drilling this joint, the whole of the operations in connection with the building of the remainder of the river span were reviewed. It was necessary, for reasons to be given later, to lift the projecting ends of the river span when the first inclined front ties were fixed, and again when a second set of front ties had been attached to the main girders. Each of these operations would rock the base of the towers backwards, though the deflection of the main girders during building would again move them forward to some extent; further, the tops of the towers would move steadily forward, due to the elongation of the anchor-ties, to the shortening of the bottom boom of the land spans, and to compression of the towers themselves. If the towers were started with an even bearing upon the end posts and in line with them, the various motions referred to would have thrown the whole of the stress upon the front limb of the towers when they were carrying their maximum load, and would have induced heavy shearing stresses in the diagonal bracing between the limbs. From these considerations it was evident that the towers should be so set originally that when carrying their full load they would be sitting evenly upon their bases, and be free from any bending stresses. After careful calculations had been made of the various motions, it was decided to pull over the top of each tower 4 inches towards the land span, and as the anchor-ties had an unavoidable waviness in plan, to allow an additional ½ inch for them straightening out, making 4½ inches in all. As the bolts connecting the base of the towers to the main girders were incapable of resisting the tensile stresses due to the rocking of the towers, one-third of them at the front and back had the nuts slackened back (A on Fig. 13), leaving only the bolts through the central 3 feet firmly screwed down (B). In this way the base of the tower was given comparative freedom for any pivot action, and the tower was relieved of considerable bending stresses, while the full strength of the bolts in shear was maintained for the connection of the temporary horizontal flange-tie to the main girders.

A force of about 7 tons, applied by union screws at the closing joints, was required to pull each tower over to the full extent. The bases of all the towers opened $\frac{1}{4}$ inch at the front edge, tapering back for 3 feet to where the bolts were firmly screwed down; there was no apparent distortion of the vertical side plates of the towers, beyond a slight permanent curvature of the bottom edge.

On the completion of stage 2 of the erection, each of the lower lengths of the first inclined ties was connected to a large gusset riveted to the side plates of the bottom boom of a main girder (Figs. 8, Plate 3); they projected several feet above the top booms, leaving a gap of about 18 inches to the upper lengths, which had been erected downwards from the gussets at the top of the towers. At this point it was necessary to take the whole load from the overhanging portion of the span on to the inclined ties, and to relieve the temporary ties at the end posts of all stresses to enable them to be cut out. The removal of these temporary ties gave the requisite freedom for the tops of the end posts to move backwards, since the projecting ends of the main girders had to be raised to a sufficient height to allow for their further deflection in building them to the centre of the span. As the temporary ties at the end posts were comparatively short and stiff, the process of lifting the ends of the main girders the requisite amount would have thrown them into compression, and with the locked bearings at the bottom, it would have been necessary to bend or deflect the entire girder upwards to get the required lift at the projecting ends. The calculated deflections of the main girders during building indicated that the projecting ends should be raised 4 inches at this point, in order to have the required camber when completed, and the lift was made in two stages. In the first stage it amounted to $1\frac{1}{16}$ inch, being made by means of a special hydraulic gear acting on the inclined ties at the gap. A force of 200 tons on each pair was estimated as sufficient to relieve the end-post ties of all stress. The ties were now severed by first cutting out all rivets connecting the temporary horizontal ties to the flange-plates of the land spans, and then by drilling through each of the vertical side tie-plates in two lines close to the end posts of both spans.

From careful measurements made before and after cutting out the ties, it was found that the tops of the end posts came together between 0·02 and 0·04 inch and that the end posts of the river span dropped $\frac{1}{16}$ inch below those of the land spans. This drop in the end posts was observed to take place after the majority of the holes had been drilled, and indicated that the ties had been carrying a large shearing stress either from the building or lifting operations.

The outer ends of the main girders were now raised 3 inches, and the cover-plates at the gap in the inclined ties were drilled and bolted on. During this lift the top of the end posts of the river span moved backwards between 1·12 inch and 1·15 inch, and the top of the towers moved forward between $\frac{3}{8}$ inch and $\frac{1}{2}$ inch. The opening between the flange-plate and at the front of the base of each tower practically closed up, leaving the latter sitting fairly evenly upon the girder. The steel packing-blocks at the bottom of the end posts showed a total vertical motion of $\frac{1}{8}$ inch at the lubricated surfaces, and opened 0·036 inch at the lower edges.

Stage 3.—When the third stage of the erection was completed, the second temporary ties were built in place and a gap was left at the tie-joint as before. The actual deflection at this point agreed exactly with the estimated amount of 2·2 inches in all cases, and was accepted as a confirmation of the calculations. The hydraulic stressing-gear was fixed at the gap, and the ends of the main girders were raised $1\frac{5}{16}$ inch by a force of 380 tons acting on each pair of ties, which was just sufficient to relieve the first inclined ties of all direct stress. It was computed that if these first ties were just relieved of stress, the elongation of the second temporary ties, during the building of the remainder of the span to the centre, would again cause them to carry a part of the load, notwithstanding the slight hogging of the main girders. The second inclined ties were designed to take the whole weight, as the original intention was to cut out the first ties after the second set had been erected and stressed. Further consideration, however, showed that nothing was to be gained by removing these ties and that, by simply relieving them of stress and allowing them to come into action again, the deflection of the girders at the centre would be materially reduced. The actual maximum deflection of the main girders at the junction of the second ties to the bottom booms, when the girders were built to the centre, was $\frac{3}{16}$ inch less than the amount calculated on the assumption that the first inclined ties were removed.

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During the third stage the opening at the bottom of the thrust-blocks closed up and transmitted the thrust evenly over the section of the bottom booms, and the base of each tower was observed to have eased slightly at its front edge. The observations made after the lift was completed at the second ties showed, that the vertical motion at the lubricated surfaces of the thrust-blocks was $\frac{5}{16}$ inch, and that they had again opened up at the bottom edges between 0·025 inch and 0·03 inch. The top of the end posts had moved backwards about 0·36 inch and the top of the towers had moved forward another inch, making $2\frac{3}{8}$ inches since they were set.

Stage 4.—The fourth stage of the erection (Figs. 14, Plate 4) up to the closing joint in the main girders was rapidly completed. The length of the plates and angle-bars of the closing pieces varied from 8 feet to 20 feet according to their position, and the measurements for them were made by rods of yellow pine, $1\frac{1}{2}$ inch square. These were marked off on the centre of the plate or bar and cut to the exact length, one rod being used for each plate or bar.

During the progress of the erection the variations due to temperature were recorded, as it was anticipated that large movements both vertically and horizontally would take place at the projecting ends of the girders, due to the rapid absorption and radiation of heat by the long plate-ties. These observations were made for guidance in drawing up the programme of operations for putting in the closing lengths of the main girders. When these girders had been built out to the centre closing-length, the maximum movements from a temperature-range of 10° F. in the shade, and of 17° F. in the sun, were $\frac{5}{8}$ inch vertically and $1\frac{3}{8}$ inch laterally, of which $1\frac{1}{4}$ inch was up-stream or westwards from the morning sun, and $\frac{1}{8}$ inch down-stream from the sun in the afternoon. The average movements, however, on a dull day were $\frac{1}{2}$ inch vertically, and $\frac{1}{2}$ inch laterally from a shade temperature-range of 8° F., and the 8-foot gap at the centre closed about $\frac{3}{8}$ inch in the bottom boom and $\frac{9}{16}$ inch in the top boom. The maximum expansion occurred between 2 p.m. and 3 p.m., and the girders moved laterally away from the sun. From these observations it was evident that the measurements for the closing-lengths would have to be taken when the temperature was a little above the mean of the previous days, to give a reasonable opportunity for erecting the closing-lengths while the girders were contracted. The measurements were taken about midday on a dull day when the temperature was 59° F., the time occupied being about $\frac{1}{2}$ hour.

The length-rods were despatched to the works in Glasgow, where wooden templates were made of each bar and plate in the closing-lengths. Accurate template models had been kept of the closing-joints, and these enabled the separate angles and plates of the booms to be cut to the exact size, and all holes in them to be drilled to correspond with the holes in the joints at the site. The closing-lengths of the bottom booms were made $\frac{1}{8}$ inch shorter than the measured lengths, to ensure the sections of the top booms butting at the closing-joints before those in the bottom booms. An interval of 5 working-days elapsed from the time the measurements were taken until the closing-lengths were completely erected in place, the method being to bolt up one end ready for drawing together and bolting the closing-joint. The 10-ton crane on the north side was dismantled before the closing-lengths were built on that half of the main girders, but the crane on the south side of the joint was left to complete the erection, and to equalize as far as possible the weights and deflections of each half of the bridge. The levels of the corresponding halves of the girders agreed within $\frac{3}{8}$ inch, and the centre lines coincided exactly. The ends of the flange-plates on the completed-lengths of the girders projected over the vertical webs of the closing lengths, and were loosely bolted and sprung back from the web-plates near the closing-joints, allowing for any unequal vertical motion of the corresponding halves from temperature.

When everything was ready no delay occurred in the closing-operations, as the temperature conditions were favourable. The early-morning temperature was 49° F., and the abutting faces at the closing-joint were $\frac{1}{4}$ inch open in the top booms. By noon the temperature had risen to 61° F., and the top-boom joints were butting hard on each other, while the closing-joints of the bottom booms were about $\frac{3}{16}$ inch open. This practically corresponded with the amount by which they had been made shorter in length. The closing-joints in the bottom booms were first secured. Barrel drifts, having a maximum diameter of $\frac{3}{4}$ inch, were inserted into the holes at the joints, and each drift was hammered up in succession, gradually drawing the joint together until $\frac{3}{4}$-inch bolts could be inserted in the holes. The drifts were rapidly replaced by others of $1\frac{1}{16}$ inch maximum diameter, the full size of the holes, and each was hammered up in succession until the joint was drawn up tight and all the holes had been filled with full-sized bolts. The top-boom joints were then bolted up and secured without any difficulty. The riveting of the angles and vertical web-joints followed immediately upon these operations, which occupied about an hour. Provision had been made to push the girders forward or draw them backwards, and to raise or lower them, if the movements due to temperature did not act as desired.

As the river span was now continuous with the two land spans at each end, a new provision for temperature-movements was required. In deciding the positions of the fixed and expansion bearings under the different girders on the river piers, this point had been kept in view, and the permanent expansion-bearings were placed under both land and river spans on the north river pier. These bearings, which had been locked in position during the erection of the river span, were now released, and the girders were rendered free to move longitudinally over this pier and also over the middle pier under the two north land spans. The falling temperature during the night caused a longitudinal motion of $\frac{5}{16}$ inch at the end of the land span on this middle pier.

The total suspended weight at the time of closing the centre joint was 2,780 tons, including the temporary work and cranes. Such permanent work as could safely be omitted during the erection was left out, thus reducing the weight to be supported by the temporary ties. All the longitudinal stringers and floor-plates on the railway-floor were omitted, and all the longitudinal stringers, the floor-plates, and each alternate cross girder of the roadway-floor after the first 40 feet from the pier. The footway troughing and parapets were left out for the whole length of the span.

At the time of making the junction at the centre, the top of the towers had moved forward an additional $\frac{1}{4}$ inch, or $2\frac{5}{8}$ inches in all, since the original setting.

Temporary Reinforcement of Permanent Work.—Temporary diagonal bracing, to transmit the wind-stresses to the piers, was placed between the bottom booms where the roadway-floor plating was omitted. This bracing was made $\frac{1}{2}$ inch shorter than the required length and was drifted up tight in order to put an initial tension on it, and to ensure efficient support being given to the bottom booms while they were temporarily acting as compression members. The bottom booms were further reinforced for several bays at the end posts of both river and land spans. During the erection of the river span the diagonal and vertical web members were subjected to large reversals of stress, and all the diagonal ties, which were of flat bars, were reinforced and suitably braced to act as struts, as shown by the thick lines in Figs. 14, Plate 4. The riveting of the permanent steelwork followed rapidly on the erection, and before anything was done to the temporary ties all the erected steelwork back to the pier was completely riveted up.

Bolting Temporary Ties.—The large gussets for the temporary tie connections were placed so as to intercept the shear from the two web-systems of the main girders, and were riveted to the side plates of the bottom booms; but all joints in the temporary ties themselves were made with turned steel bolts $1\frac{1}{16}$ inch in diameter. These bolts, of which about 20,000 were used, were turned to gauge to suit the holes and made a hard driving fit. Parallel rimers were passed through all the holes in the joints after the ties were erected, and the bolts were fitted by skilled mechanics. The workmanship and fit of these bolts was equal to the best engine-work.

Previous to deciding to bolt the joints, two tests of riveted and bolted joints were made under conditions similar to those obtaining at the site. In the first Table of Appendix II are recorded the comparative tests carried out on a hydraulically riveted joint and one made with turned steel bolts of a driving fit. The holes were drilled the full size through the flats, and the rivets were put in under similar conditions to the ordinary shop-work. The steel bolts were taken at random from the heap. The holes for the bolts and rivets were not rimered out, but the bars were taken apart and cleaned in the usual way. The flats were of mild steel and the rivets and bolts of "rivet steel."

In the two tests made the two end bolts sheared almost simultaneously, and the third bolt had the upper section displaced $\frac{1}{32}$ inch beyond the lower section, being on the point of shearing. The end rivet in both tests had the upper section displaced about $\frac{1}{32}$ inch beyond the lower section, and had practically failed when the bolts sheared. No slip was observed at the bolted joint until the shearing stress exceeded 7 tons per square inch, nor at the riveted joint until the stress exceeded 11 tons per square inch.

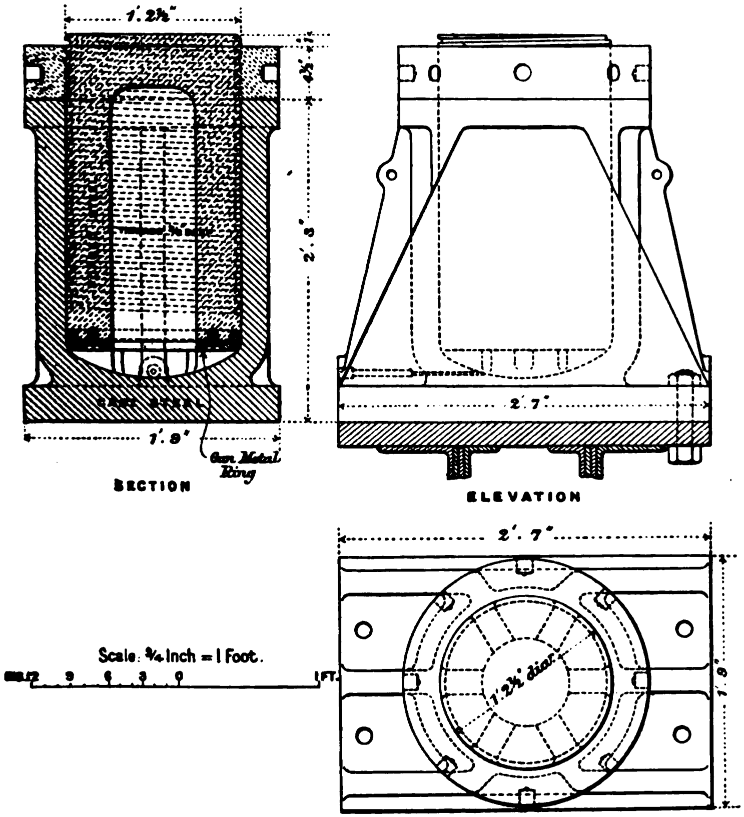
A third test was made in which black wrought-iron bolts were used $\frac{1}{16}$ inch smaller in diameter than the holes. The result of this test is given in the second Table of Appendix II. In this case no slip at the bolted joint was observed until the shearing stress exceeded 6 tons per square inch.

These tests demonstrated the high efficiency of a bolted joint made in the manner described, and showed that a riveted joint had little advantage in strength, while the bolted joint had the great advantage of being more rapidly dismantled when the temporary work was being removed. There was, further, the certainty that all bolts were filling the holes and doing their full share of the work, in a better manner than would rivets which had been put in under the conditions obtaining at the site.

The small superiority in strength of the riveted joint was, no doubt, due to the greater frictional adhesion between the plates from the contraction of the rivets in cooling, and emphasizes the importance in a bolted joint of having all bolts screwed up as hard as possible. The tests indicate that under ordinary working conditions the bolts or rivets in a joint are not called upon to resist shearing stress. To see how far this was true in the actual ties, some of the bolts were taken out while the tie had the maximum stress in it; these bolts were easily hammered out and had no indication of any distortion or marks due to shear.

Hydraulic Stressing-Gear.—The general arrangement of the hydraulic stressing-gear for each pair of main temporary ties is shown in Fig. 12, Plate 4. A lower cross girder was securely bolted to the two main ties about 4 feet below the gap in the closing joint, and an upper cross girder was fixed about 7 feet above the gap. Two plate ties, each formed of four plates, 10½ inches by ¾ inch, riveted together, were placed between and riveted at their lower ends to the webs of the lower cross girder; then passing between the webs and through the flanges of the upper cross girder, were riveted at their upper ends to a crosshead girder, which was free to move between the two main ties. On account of the relatively great thickness (3 inches) of these two side ties, it was considered advisable to arrange for ensuring an even distribution of the stress on each plate and for avoiding a bending stress on the connecting rivets; they were therefore made to project beyond the two girders at each end and were riveted together outside their connections to the webs of these girders. The upper cross girder formed a seating for two hydraulic screw-jacks, which were placed between it and the crosshead girder. On the rams being brought into action, the crosshead girder was pushed upwards, and by means of the connecting ties pulled up the lower cross girder, and so raised the main girders of the bridge. The jacks were coupled together on the same hand-pump so that the hydraulic pressure in each would be equal. These hydraulic jacks are of special design, and are shown in *Figs. 15.* The rams were of forged steel, 14½ inches in diameter, and upon them was cut a screwed thread, which engaged in a nut where they projected through the cylinder. As the pressure forced out the rams the nuts were gradually screwed back, and kept continually bearing upon the top of the cylinders, so that in the event of any failure of the hydraulic pressure there was no possibility of the rams running back, and further the rams could be securely held in any desired position. This stressing-gear was perfectly safe and reliable in its action, and enabled the operations of raising the ends of the main girders to be performed in a satisfactory manner. The screw device on the rams rigidly maintained the gap in the main temporary ties, until the closing-pieces had been drilled and securely bolted on. After this was done an additional force of about 18 tons on each jack was required to release the pressure on the nut, and allow the rams to be run back for removal of the gear. By means of this gear the calculated stress on the temporary ties could be confirmed at each operation of raising the girders.

Figs. 15.


1'. 2½"
4½"
2'. 8"
Cast Steel
1'. 9"
Gun Metal Ring
SECTION
2'. 7"
ELEVATION
2'. 7"
1'. 2½" dia.
1'. 8"
PLAN
Scale: ¾ Inch = 1 Foot.
1 FT.

Details of Hydraulic Jack.

Relieving Temporary Ties.—After the closing joint in the booms at the centre of the span was made, the remainder of the permanent steelwork in the floors was erected and riveted. The final operations in connection with the release of the temporary ties were then undertaken. Two of the hydraulic screw-jacks, which had been used in the stressing-gear, were placed under each main girder of the land spans at the ends of the anchor-ties. The end of each span was then raised 11 inches by a force of about 720 tons. The centre of the river span lowered 2⅝ inches when the lift of the end of the land spans was 10 inches. The additional lift of 1 inch entirely relieved the temporary ties of all stress, which was clearly indicated by their assuming the original waviness in plan that they had on erection, and by no further deflection taking place in the river span. The bolts in the lowest joint in each anchor-tie were then taken out, and the ends of the land spans were lowered back on to their bearings. The steel thrust-blocks between the bottom booms of the end posts were immediately removed, and the whole of the temporary work was rapidly dismantled. The distance between the lower end of an anchor-tie and the gusset connecting it with the bottom boom of the land span was measured after the span was back on its bearings, and the gap was found to be 5 inches. The top of each tower had moved forwards about 3 inches or a total of 5½ inches from the original setting.

The large temporary gussets for the attachment of the temporary ties to the bottom booms of the main girders were not removed, but were cut off flush with the tops of the side webs by means of the oxy-acetylene blow-pipe. The time taken to burn through a plate 14 feet wide by 1¼ inch in thickness was 50 minutes.

Deflections and Stresses during Erection.—The deflections were computed by the principle of work, checked graphically by displacement diagrams. The modulus of elasticity was assumed to be 12,000 tons per square inch, and a reduction in the calculated deflection was made for the stiffness of the riveted joints, covers and gussets, amounting to 20 per cent. of the deflection contributed by the main girders, and to 10 per cent. of that contributed by the temporary ties. The agreement between the actual and calculated deflections confirmed these assumed values.

The maximum permissible unit stresses in the permanent and temporary work during erection were carefully considered before elaborating the scheme, and it was decided in consultation with Mr. C. A. Harrison, the engineer for the bridge, that the stresses should not exceed one-fourth of the ultimate strength under the dead load, and that under a wind-pressure of 50 lbs. per square foot the combined stresses from the dead and wind loads should not exceed one-half of the elastic limit.

These values are low for direct stresses in temporary work, but it must be remembered that there are many accidental secondary stresses which it is impossible to foresee and to provide for, and a relatively small amount of additional material is required to make the work quite safe. The calculations for the loads and stresses during the various stages of the erection are shown in Appendix III.

Considerable care was manifested throughout to avoid large secondary stresses in either the temporary or the permanent work. The towers and temporary ties were set out at such inclinations that they should be, as far as possible, in the direct line of stress when subjected to the maximum load, and the observations made during the operations confirmed the original assumptions. The adoption of pin connections or their equivalent, at the ends of the temporary ties and at the base of the towers, would no doubt have removed any anxiety from this cause; but the success which attended the operations throughout leaves this point open to doubt.

About 700 tons of steelwork, 4,500 cubic feet of timber in the trestles, and 3,900 cubic feet in the platforms and gangways, was used in the temporary work; or approximately 5·4 cwt. of steelwork and 3¼ cubic feet of timber per ton of permanent work erected in the river span.

The total amount of steelwork in the whole contract was 8,440 tons.

When the closing joints in the river span were riveted up, all the permanent work omitted during the erection was put in place and permanently riveted, the temporary work being dismantled. This was practically completed by the end of December, 1908, and the roadway and permanent way was finished early in 1909. The erection of the river span took 10 months.

The bridge was tested by Colonel Von Donop, acting for the Board of Trade, on the 3rd June, 1909. Twelve locomotives, having a total weight of 1,190 tons (3·34 tons per lineal foot on the double track), were run over the bridge. The maximum deflections of the main girders of the land and river spans were, respectively, ¾ inch and 1⅝ inch.

The whole contract took about 4 years to complete, and the total cost for the bridge and approach-railways was about £325,000.

The bridge was opened by the Earl of Durham, K.G., on the 10th June, 1909, and by the gracious permission of Her Majesty the Queen it was named the Queen Alexandra Bridge.

The bridge and approach-lines were carried out from the designs and under the supervision of Mr. C. A. Harrison, D.Sc., M. Inst. C.E., Chief Engineer to the North Eastern Railway, whose Resident Engineers were Mr. P. Bulmer, M. Inst. C.E., and Mr. F. C. Buscarlet, Assoc. M. Inst. C.E. The superstructure of the main bridge was designed by Mr. C. E. Procter, and the 212-foot skew bridge by Mr. G. F. Jackson, Assoc. M. Inst. C.E., both of the Company's engineering staff.

The contract for the whole of the work was undertaken by Sir William Arrol and Company, of Glasgow, under the personal supervision of Mr. A. S. Biggart, Assoc. M. Inst. C.E. Mr. R. C. McDonald was the contractors' agent at the site for the steelwork. Messrs. Mitchell Brothers were the sub-contractors for the masonry and earthwork. Mr. J. Mitchell Moncrieff, M. Inst. C.E., acted as Consulting Engineer to the contractors, and Mr. Adam Hunter, M. Inst. C.E., prepared and supervised the scheme of erection for them.

The Paper is accompanied by twenty-two sheets of drawings and three Tables, from which Plates 3 and 4, the Figures in the text, and the Appendixes have been prepared. There is also a portfolio of photographs.

[Appendixes.

APPENDIXES.

APPENDIX I*—PARTICULARS OF SINKING OF CAISSON.

1	2		3	4		5	6	7	8	9
Date.	Depth below River-bed at End of Period.		Mean Weight on Cutting-Edge during Period.	Air Pressure, Lbs. per Square Inch.		Strata Encountered.	Number of Sinkers.	Men-Hours.	Material Excavated (Measured from Skips of 8 Cu. Ft. Capacity).	Volume per Man per Hour.
	Feet.	Ins.	Tons.	Max.	Min.				Cub. Yds.	Cub. Yds.
1905.										
Oct. 24 and 25[1]	3	0	364	..	..	Mud				
Oct. 26–Nov. 13[2]	6	2¾	1,053	..	..	Yellow sand				
Nov. 14–Dec. 8[3]	19	0½	2,596	12	2	„ „	11 to 12	2,310	1,400	0·605
Dec. 9–Feb. 5	34	4⅛	4,703	19	8	Sandy clay	10 „ 12	2,108	1,045	0·50
1906.										
Feb. 6–May 28[4]	59	3⅛	7,429	30	15	Stiff grey clay	12 „ 16	5,014	2,958	0·59
May 29–June 16	65	3	9,292	33	28	Yellow sand with lumps of sandstone	16	1,520	947	0·62
June 18–July 5	70	9⅞	9,788	36	30	Seggar and grey sandstone	18 „ 19	1,830	570	0·31
July 6–July 24	72	3	9,890	39	33	Laminated red sandstone	18 „ 24	1,285	464	0·36
August 13	72	3¼	9,890	..	..	„ „ „				
160 working days.							Totals	14,067	7,384[5]	

* This Table was added after the discussion took place.

[1] River-bed previously excavated to a depth of 2 feet over site for caisson (165 cubic yards).

[2] Maximum amount out of level at lowest corner during sinking was 18⅜ inches on 13th November.

[3] Excavation under air-pressure started 14th November.

[4] Whole of caisson below river-bed on 20th February.

[5] The actual volume displaced by the cutting edge during sinking was only about 5,945 cubic yards.

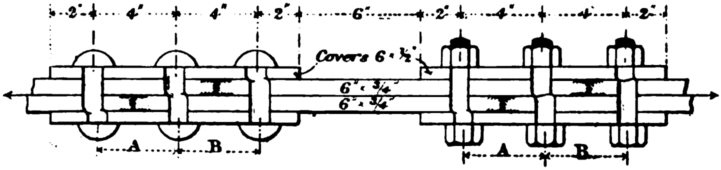
APPENDIX II.

Comparative Tests on the Shearing Strength of Steel Rivets and Turned Steel Bolts. (*Fig. 16.*)

Three bolts 1$\frac{1}{16}$ inch diameter in drilled holes 1$\frac{1}{16}$ inch diameter.

Three rivets 1$\frac{1}{16}$ inch diameter hydraulically riveted in drilled holes 1$\frac{1}{16}$ inch diameter. Shearing area of rivets and bolts, 2·66 square inches.

Fig. 16.



To measure the slip centre-punch marks were placed on each side of the joints, and measurements were taken by dividers at each load given and measured on a finely divided scale.

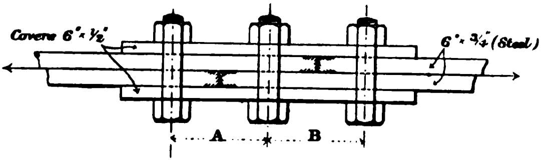
Total Load.	Shearing Stress.	Slip (in Inches).				Remarks.
		Rivets.		Bolts.		
		A	B	A	B	
Tons.	Tons per Sq. In.					
14·8	5·6	..	..	..	..	No slip.
19·7	7·4	..	..	$\frac{1}{32}$	$\frac{1}{32}$	
24·6	9·3	..	..	$\frac{1}{32}$	$\frac{1}{32}$	
29·5	11·1	..	..	$\frac{1}{16}$	$\frac{1}{16}$	
34·4	12·9	$\frac{1}{16}$	$\frac{1}{16}$	$\frac{3}{32}$	$\frac{3}{32}$	
39·3	14·8	$\frac{3}{32}$	$\frac{3}{32}$	$\frac{1}{8}$	$\frac{1}{8}$	
44·2	16·6	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{3}{16}$	
49·2	18·5	$\frac{7}{32}$	$\frac{7}{32}$	$\frac{1}{4}$	$\frac{1}{4}$	
52·8	19·9	..	..	..	..	Bolts sheared

N.B.—In a former test the bolts sheared at 19·4 tons per square inch.

TEST ON THE SHEARING STRENGTH OF BLACK WROUGHT-IRON BOLTS. (*Fig. 17.*)

Three bolts 1 inch diameter in drilled holes $1\frac{1}{16}$ inch diameter. Shearing area of bolts, 2·36 square inches.

Fig. 17.



Total Load.	Shearing Stress on Bolts.	Slip (in inches).		Remarks.
		At A.	At B.	
Tons.	Tons per Sq. In.			
4·9	2·1	..	..	No slip.
9·9	4·2	..	..	No slip.
14·8	6·3	$\frac{1}{32}$	$\frac{1}{32}$	
19·7	8·4	$\frac{1}{16}$	$\frac{1}{16}$	
24·6	10·4	$\frac{7}{64}$	$\frac{7}{64}$	
29·6	12·6	$\frac{9}{64}$	$\frac{9}{64}$	
34·4	14·6	..	..	
39·3	16·7	..	..	Bolts sheared.

APPENDIX III.

CALCULATIONS FOR LOADS AND STRESSES DURING ERECTION OF RIVER SPAN.

In all the calculations wind-pressure has been considered as acting on the bridge at an angle producing a horizontal component of 50 lbs. per square foot, and a downward vertical component of 5 lbs. per square foot of exposed area. The exposed area of the windward girder has been increased by 50 per cent. to make allowance for the leeward girder; and in the case of the temporary ties and tower by 100 per cent.

Stage 2 (Figs. 14, Plate 2).—In this is shown the first portion of the 330-foot span erected and supported by ties at the top boom of the main girder; the Figure also shows the temporary work in position preparatory to the erection of the second portion of the span.

Dead load . . . . = 747 tons.
Moment about end bearings 747 tons × 34·6 feet = 25,846 foot-tons.
Dead load on supporting ties at the top boom—
For two main girders 25,846 ÷ 27 feet . . = 958 tons.
Maximum load on supporting ties (for one girder only)—
Dead load = $\frac{958}{2}$ . . . . = 479 "
Wind load vertical component . . . . . . = 4 "
Wind load horizontal component . . . . . = 51 "
Total = 534 "

Section of supporting ties (one girder)—
Four plates 24 inches × ⅞ inch − 6*r* . . . . = 63·0 sq. ins. (net).
One plate 68 inches × ¾ inch − 10*r* . . . . = 42·5 " " "
Total = 105·5 " " "

Dead load stress = 479 ÷ 105·5 . . . . = 4·6 tons per sq. in.
Maximum stress = 534 ÷ 105·5 . . . . = 5·1 " " " "

Stage 3.—Shows the first and second portions erected and supported by ties A and C; the Figure also shows tie B erected preparatory to building the third portion of the span.

Dead load (total) . . . . = 1,126 tons.
Moment about end bearings . . . . = 63,844 foot-tons.
Maximum load on tie A (one girder)—
Dead load . . . . = 557 tons.
Wind load vertical . . . . = 6 "
Wind load horizontal (transferred) . . . . . = 113 "
Total = 676 "

Section of tie A—
Four plates 24 inches × ⅞ inch − 2*r* . . . . = 76·5 sq. ins. (net).
Dead load stress . . . . = 7·3 tons per sq. in.
Maximum stress . . . . = 8·9 " " " "

Stage 4.—Shows the 330-foot span erected to the centre and supported by ties A, B and C.[1]

Dead load (total) . . . . = 1,391 tons
Moment about end bearings . . . . = 108,219 foot-tons.
Maximum load on tie B (one girder)—
Dead load . . . . = 715 tons.
Wind load vertical . . . . = 17 "
Wind load horizontal (transferred) . . . . . = 124 "
Total = 856 "

Section of tie B—
Eight plates 24 inches × ⅝ inch − 2*r* . . . . = 110 sq. ins. (net).
Dead load stress . . . . = 6·5 tons per sq. in.
Maximum stress . . . . = 7·8 " " " "

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[1] In calculating the load on tie B it is assumed that tie A is entirely relieved of stress.

Tower.—Maximum load for one girder only—

Dead load = 695 tons.
Weight of tower = 40 „
Wind load = 137 „
Total = 872 „

Section of tower = 189 sq. ins. (gross).
Dead load stress = 3·9 tons per sq. in.
Maximum stress = 4·6 „ „ „ „

Steel Packing Blocks at Bottom Booms.—Maximum compression at bottom boom of main girder—

Dead load = 550 tons.
Wind stresses = 345 „
Total = 895 „

Tie C.—Maximum load for one girder only—

Dead load = 600 „
Wind load = 118 „
Total = 718 „

Section of tie C—
Four plates 30 inches × $\frac{7}{8}$ inch − $2r$ = 97·1 sq ins. (net).
Dead load stress = 6·2 tons per sq. in.
Maximum stress = 7·4 „ „ „ „

200-foot Land Span.—Total weight, complete = 1,484 tons.
Load on each girder bearing 1,484 ÷ 4 = 371 „
Upward pull from tie C, dead load only = 239 „
Difference = 132 „

Discussion.

The Chairman. The **Chairman**, in moving that the thanks of The Institution be accorded to the Authors for their important and interesting Papers, thought the members would agree that they were not only valuable as records of works of great magnitude, but also as opening out a wide field for a discussion, in which he hoped that those who had had experience in the construction and erection of bridges would take part.

Mr. Matheson. Mr. **Matheson** exhibited a series of lantern-views illustrating the progress of the work and the finished structure, and expressed his obligation to the members for their kind reception of his description of the Clyde bridge. It might be thought that the Paper was rather long, but much of it was devoted to description of the deep foundations. It was not claimed that the method which had been adopted in constructing these foundations was in any way new, but looking through the Proceedings of The Institution and the Transactions of other societies he had not been able to find any detailed description of the method, and that was the reason why the deep-foundation work of the Clyde bridge had been described at such length. Apart from the structural work, the most outstanding physical feature of the bridge was its great width. Although it was only about 700 feet long, its area was upwards of 2 acres. That area was needed for the station-yard which it was thought necessary to construct in the immediate precincts of the train-shed of the station, and, while the station-yard was certainly spacious, it was considered necessary to have it of such a size because it was believed that a spacious and convenient station-yard was the most important adjunct of a great terminal station, an opinion which the use of the Central Station yard had fully justified. It might be suggested that it was not economical to build such a wide bridge over the river for the purposes of a station-yard, and no doubt the yard had cost a large amount of money; but those who knew Glasgow well would recollect that even if the station-yard had been constructed clear of the river, either to the north or to the south, it would not have been less costly, because on either side of the river the land was very expensive. Members might be surprised to know that the Caledonian Railway Company had had to pay to the Clyde Navigation Trustees as much as £75,000 for wayleave, in respect of the new bridge, crossing over the river and quays. That was at the rate of

Mr. Matheson.

about £10 per square yard of the area over the river and quays. It seemed a very high price for wayleave, but had the station-yard been constructed on the north side or on the south side of the river, the cost of the land per square yard which would have been required would have been much higher. Had the station-yard been constructed on the north side of the river, the land would probably have cost at least four times as much per square yard as had been paid for wayleave over the river. Being over the river, the foundations of the substructure of the station-yard, that was the bridge, were of necessity costly. While the substructure of a similar station-yard clear of the bridge would not have been so costly, the land would, as had been indicated, have been more costly, and therefore from the point of view of the economics of construction it had really been immaterial whether the station-yard was constructed over the river or clear of the river. In the one case the land was expensive; in the other case the works were expensive.

Mr. Buscarlet.

Mr. **Buscarlet**, in exhibiting a number of lantern-slides illustrating the construction of the Wear bridge, remarked that the fact that the Queen had permitted her name to be given to the bridge testified to its importance. The work had been carried out with future requirements as well as present necessities in view. Throughout the execution of the work, which had occupied nearly 4 years, there had been only one fatal accident, and that was due without doubt to drunkenness.

Dr. Harrison.

Dr. C. A. **Harrison** remarked that the Wear bridge was a work of great interest for various reasons. One reason was that, as far as he could find out, the Wear bridge and the old high-level bridge across the Tyne, which was erected by Robert Stephenson and his (the speaker's) uncle, Thomas Elliot Harrison, were the only two double-deck bridges in England. On the table were the drawings of the old high-level bridge, belonging to the Library of The Institution, and it would be seen that that bridge was built with cast-iron arches on which were cast-iron standards carrying the railway. From the same arches there hung very heavy bars, which extended down below the roadway, and on them the roadway was carried. That was rather different from the new bridge, but in his opinion it was the finer bridge of the two. The new bridge also had to carry a road and railway, but circumstances had altered in the 50 years since the old high-level bridge was built, and it was necessary to have two very heavy side girders to carry the railway on the top and the roadway at the bottom, besides which the cross girders had to project outside the main girders to carry the footpaths and gas- and water-pipes in the neighbourhood. Some silly statements about the bridge

Dr. Harrison. had been made in the newspapers, one account saying that the new bridge was larger than the Forth bridge, which of course was nonsense. It arose from the fact that the centre span, erected by the cantilever process, was heavier than the 300-foot suspended spans of the Forth bridge. The main girders weighed 950 tons each, or 475 tons projecting on each side, which was a very considerable weight. Probably it would be asked why, when the cantilever principle was used at the finish, a cantilever bridge had not been made. The bridge was the result of a certain amount of negotiation between the Sunderland Corporation and the North Eastern Railway, the Corporation paying a portion of the cost. A saving to both the Corporation and the Railway Company had resulted from building a joint bridge, instead of a railway-bridge in one place and a road-bridge in another. Twenty-five years ago, when a railway-bridge was built across the Wear, permission was given to lessen the waterway to a very appreciable extent, and he had thought he could do the same with the present bridge; but after the new bridge had been designed and had been passed by the Wear Commissioners and by the Corporation, he was told that not a single timber must be put into the river, because there was a shipyard on either side and no risk could be run of a ship launched from one of those yards running into the bridge and the Commissioners having to bear the brunt of the consequences. Accordingly, as the design of the bridge was settled, a method of building it had to be devised, and he stated in the specification one or two methods by which it might be built, but added that should the contractor find a better way of which he could approve, it might be adopted. The result was that Sir William Arrol finally decided to build the centre span out on the cantilever principle, and the credit for that part of the work was due to the contractor. One rather interesting point, apart from the building of the bridge, was that the level of the rock on the south side was 50 feet higher than that on the north side, although the river was only 340 feet wide altogether. On the south side excavation was saved to the extent of about 16 feet, but on the north side it was necessary to go down 70 feet, and then only shale was reached. The most interesting part of the whole work was the building out, and the greatest interest no doubt lay in the joining-up. A day was selected some months before, on which all the templates for the joining-up portion were prepared and sent to Glasgow; and, as luck would have it, upon the day of the joining-up the temperature only differed about 1° F. from the day on which the templates were taken. The result was very gratifying to everybody. Unfortunately, owing to another engagement, he had not been able to be present, but he understood that at the crucial moment the sides expanded exactly to the right extent, and then the bolts were put in and went home as though they had been riveted on the spot. After that it was easy work to take out the bolts one by one and rivet up permanently. He had had the good fortune to be the designer and engineer for two of the largest bridges built since the Forth bridge, but he was still luckier in having had the assistance of very capable men in his own department and in both cases very efficient contractors. Dr. Harrison.

Mr. Moncrieff Mr. J. M. MONCRIEFF observed that, having had some slight connection with the Wear bridge, as Messrs. Arrol's consulting engineer for the erection, he would like to make a few remarks. The description of the erection referred in more than one instance to the changes of form which took place in the members of the temporary erection-gear, and he would like to point out that that was really a very important matter to consider in connection with all bridges of such dimensions and weight. The calculation of the ordinary direct stresses, although tedious and perhaps very lengthy at times, was a comparatively simple matter. It was a totally different thing to predict to what degree and in what directions the deformations might take place during the actual erection of the bridge. The importance of considering those deformations would perhaps best be grasped by consideration of the effect upon the large erection-towers immediately over the end posts of the main girders. If the deformations had not been considered, and if provision had not been made in the working-out of the scheme to meet the deformations, then these erection-towers, instead of being uniformly stressed over the whole of the section, would have been very heavily and possibly dangerously stressed on one edge. Again, in the short ties which were used in the earlier part of the erection at the top of the end posts of the river span, connecting them with the end posts of the shore span, it was a very interesting study to consider how the deformation under stress would be affected by the shortness of the tie, and when that was under discussion he believed Mr. Hunter made tests on india-rubber models to see whether the idea that the back end rivets would get little or no stress was true or not, and he would probably be able to say that that was actually found to be the case—that the rivets which were nearest to the centre-line between the two end posts were practically carrying the greater part of the load and those at the hinder end were taking nothing at all. Under those circumstances the idea of a uniform stress-distribution calculated at so many tons per square inch was practically useless.

Mr. Moncrieff. The heel-blocks were also somewhat interesting; if they had not been provided with proper lubrication and made as nearly free as was possible, a very severe bending stress would have been thrown upon the bottom booms and upon the ends of the girders which abutted upon each other. For instance, in the Paper figures were given to show that the compression from dead load alone between one girder and another was about 550 tons. The half-width of the end posts was 4½ feet. If no movement had been permitted, or if the coefficient of friction should reach unity, there would therefore have been a bending-moment of about 2,500 foot-tons, which, of course, would have been a very serious matter. The lubrication did not entirely get rid of that, but it was a very great relief to feel that the blocks did actually move during erection with much more freedom than was originally expected. The blocks were made in several pieces so as to have several surfaces, reliance not being placed merely on two surfaces such as would have been obtained by one single block, as that might have been dangerous. The aim was to have different surfaces, all of which had the opportunity of being lubricated and any one of which might give. The importance of that particular point was seen when the upper tie at the top of the two end posts was cut; it would be remembered that the lantern-slides had shown two lines of rivet-holes which cut the ties in two. When the first of those holes were drilled they were perfectly circular as far as could be seen, but when the metal began to get thinned down by additional holes until there was little left, quite suddenly there was an alteration, which showed that one end post went down below the other to the extent of $\frac{1}{16}$ inch and the holes became distinctly oval. The stress at that stage due to the shear must have been very considerable indeed, and was, of course, a distinct addition to the calculated direct stresses. With regard to turned bolts in comparison with rivets, the tests described in the Paper were of great interest. He had not expected that turned bolts would come out so well in comparison with ordinary first-class riveting. It was practically at his instance that Messrs. Arrol made the tests, because he doubted whether it was wise to put on ordinary turned bolts as much as would be reasonably put on first-class riveting; but the tests demonstrated quite clearly that for erection purposes it was quite safe—of course assuming that the bolts were fitted as well as Messrs. Arrol fitted the bolts in that work. The statement made in the Paper that they were practically engine-work was quite justified. With regard to the influence of the length of ties upon the permissible stress, he had already referred to that in connection with the ties at the top of

Mr. Moncrieff

the end posts. It had to be considered very carefully how the stress would be distributed over the section. It was quite a different thing when the stresses on the long inclined ties, C, from the top of the post to the heel of the shore girders, were being dealt with; they were very long, and any little irregularity in the form of those ties was compensated for by the stretch in them: one bar might be a little longer or shorter than the other without any serious detriment or danger. During the erection there had been one very anxious moment in connection with the ties, B, when carrying the very heaviest loads. Some of the ties during transmission to the site had become slightly bent, and they were too large to be straightened on the site. They were nevertheless put in place, and as the load came on the ties steadily and gradually there was considerable doubt as to whether the work should go on. However, it was decided to go on and the ties straightened out beautifully. When they were relieved of load again the one which had been longer than the other again resumed its bent form. Of course, if the ties had been short that would have been a dangerous and foolish thing to do, but with their great length the stretch due to the stress upon them compensated very largely for the inequality of length, which was really very small, although it showed largely in the bends, and this stretching justified the continuance of the loading.

With regard to the Paper on the new Clyde bridge, the Author referred to borings, particularly chisel-borings, as being proverbially fallacious. That was the common experience of engineers in general practice. From time to time bores were put down and the results were found to be not that which the bores led one to expect. But he did not think the borer could be blamed altogether, and that might be illustrated by three examples in his own experience. In one case the borer predicted that at a certain level sand would be reached, but when the piles were put down they could not be got into the sand, and it was ultimately found that it was not sand but really sandstone. Of course, if the borer had predicted sandstone and it had turned out to be sand, it might have been disastrous. In the second case there were two cylinders 12 feet apart from centre to centre with a space of only 4 feet between the cylinders themselves. The bore was put down before the two cylinders and showed soft stuff to a depth of about 80 feet below high water. One cylinder went right down to the figured depth exactly as the bore indicated, but the adjacent cylinder struck a hard bit of sandstone right across the full diameter and area of the cylinder and 15 feet above the expected level, although it was only 12 feet from the other cylinder. His resident engineer reported to him that the sand-

Mr. Moncrieff. stone was quite hard, but he instructed him to go on, and it turned out that the hard bit of sandstone was nothing more or less than an overhanging cliff of sandstone in the soft shale, and by going on the cylinder reached the same level and the same material as the adjacent one. If the work had been stopped, the cylinder would have rested on a cantilever of friable sandstone which would have been no good for transverse stress. In the third instance he had a section of ground divided up for twelve bores, forming squares practically 100 feet apart between centres. The results of the borings were so remarkable that if he had not had his own man on the ground to check everything he would never have believed they could be so regular. Nevertheless, in carrying out the work, which was dock-work, it was found that between the bores, diagonally, there was a small river-course about 20 feet below the level of the hard ground. He did not think bores should be called fallacious, but rather that they should not be absolutely trusted. He did not know how it was possible to get qualified boring contractors officially accredited by a recognized authority, as suggested by Mr. Matheson, but it might be a new labour for The Institution. With regard to surface friction on cylinders or caissons, he did not think it should be ignored entirely. It was always something to fall back upon—something on the credit side—in connection with the supporting powers of cylinders or caissons. Fixing or standardizing stresses in bridgework generally was a point to which he had referred at the first Engineering Conference held by The Institution, and Mr. Alexander Ross had since referred to the same thing in a Note discussed at a later Conference. He would suggest that some consideration should be given by The Institution to allowable stresses in structural work. They were extremely difficult to fix rigidly, but there might possibly be something better than mere guesswork. Something was known about stresses, but there was a great deal to be learned, and he thought an open discussion on stress questions would be of great benefit to every engineer engaged in structural work. In connection with the sinking of cylinders and the dangers connected therewith, Mr. Matheson referred to cutting off pressure during the sinking. He noticed that in the Clyde bridge caisson, as well as in the Wear bridge caisson, there were inclined faces in the working-chambers which would gradually brake the falling of the cylinder if the pressure were taken off. There was no harm at all in having something more than 8 feet in height in the working-chamber when it was considered that with a tall man in the chamber a fall of 4 feet would give him a terrible fright.

Mr. Moncrieff He had had cylinders come down 4 or 5 feet, and the men came out with their faces as white as paper, although they were no worse otherwise.

Mr. Cruttwell. Mr. G. E. W. CRUTTWELL considered that a wise decision had been come to in not taking the foundations of the Clyde bridge down to the rock, as the sand and gravel upon which the cylinders were founded were perfectly reliable and were at such a depth that the sand would not escape from beneath the foundations. The 70 feet to which the cylinders went was quite deep enough for the divers to work in comfortably. If the cylinders had gone down to the rock it would have meant a depth of more than 90 feet, the depth to which the neighbouring Glasgow bridge had been carried with, he believed, considerable difficulty. With regard to the skin-friction, he did not think Mr. Matheson's results were to be relied upon exactly as the actual friction due to the superficial area, because the caissons were inset a good deal; each plate was lapped inwards, so that the cutting edge forced a hole in the ground considerably larger than the space occupied by the upper portion of the caisson. Of course the ground closed in against the upper plating of the caisson as it sank, but still the loosening of the ground lessened the friction to a considerable extent. If the caisson had been built with vertical sides he believed there would have been found rather more than 4½ cwt. per square foot. At the Immingham dock, where he had been sinking brick cylinders built with perfectly vertical sides, some careful experiments had been made. The skin-friction amounted to 7 cwt. per square foot. Of course there would naturally be considerably more friction against a brick surface, although they were fairly smooth bricks, than against an iron or steel surface; yet he thought the difference would have been hardly so large as between 7 cwt. and 4½ cwt. In basing calculations upon the quantity of kentledge likely to be required, more allowance should be made than Mr. Matheson's experiments would lead one to think. With regard to the cross girders above the air-chamber of the caissons, it was not clear whether any credit had been taken, in calculating the strength of the cross girders, for the weight of the concrete placed above them, or whether they were calculated to take the full upward pressure due to the maximum air-pressure. If so, no doubt they would be very heavy, but personally he did not see why credit should not be taken, in designing deck girders, for the weight of the concrete, when that concrete had to be applied during the sinking. On beginning to sink a caisson there was very little upward pressure due to the air, but near the bottom there were very large pressures, balanced to a large extent, if not entirely, by the weight of the

Mr. Cruttwell. concrete which was piled upon the floor of the air-chamber before it reached the bottom. With regard to the sinking of the caissons, he did not know whether it would not have been a better arrangement to have had screws for lowering, instead of the links and cotters adopted at the Clyde bridge. The lowering would then be very gradual. Mr. Matheson gave the cost of the piers as £2 15*s.* 6*d.* per cubic yard of contents displaced, or, adding the cost of the caissons, £3 15*s.* 6*d.* per cubic yard. In his own Paper on the Tower bridge, he mentioned that the solid contents of the piers worked out to £2 3*s.* 7*d.* per cubic yard,[1] which was much less than at the Clyde bridge; but that, no doubt, was accounted for by the Tower bridge piers having been put in in open caissons, whereas at the Clyde they were put in under air-pressure. Probably, therefore, the difference between the two costs might be assumed to be due to the use of compressed air.

Mr. Kirkpatrick. Mr. C. R. S. Kirkpatrick felt a somewhat personal interest in both Papers because he happened to be one of those who were unsuccessful in tendering for both bridges and knew the difficulties to be encountered; but as the prices he advised should be put in the tenders were too high, possibly he had overrated the difficulties. In regard to the bridge over the Wear, he noticed that tests had been made of the crushing-stress of the red Dumfriesshire stone and the result was only 360 tons per square foot. He had made several tests of stone from the immediate vicinity of Dumfries —within a 10-mile radius—and had got results as high as 852 and 643 tons per square foot. There was no doubt, therefore, that the piece referred to on p. 61 was rather below than above the average. He noticed the use of the words "bitumen concrete" in both Papers. As the description of the material used for the Wear bridge said "pure pitch and oil and coke breeze," the term "bitumen" was hardly correct. He thought it should be applied only to the natural mineral product, and not to a composition of gas-tar pitch. It reminded him of a contractor who once told him that he intended to comply with a specification for "natural bitumen" by using a mixture of foundry-sand, pitch, and oil, adding about sixpennyworth of pure bitumen to give it a name. He noticed that in the Wear bridge the contractors used only one shaft for the caisson-sinking, and in that respect he thought they took a serious risk. Although they provided for two in the first place, for some reason or other one was abandoned. If the two had been adopted there would have been the cost of another air-lock and

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[1] Minutes of Proceedings Inst. C.E., vol. cxiii, p. 136.

Mr. Kirkpatrick.

another shaft, but he thought the contractors would have saved money in the end. It rather suggested that possibly the air-compressing plant was not in duplicate, and he would like to know if that was the case. In the sinking of the caisson no mention was made of the method of lighting it, but he had been told that candles were used and not electric light. If that was the case he would like to know whether there was any special reason why low-voltage electricity had not been used. He thought it might be taken as essential that electricity should be used, because candles were objectionable from other points of view, apart from giving insufficient light. He had found it very difficult to understand why the caisson sank 6 inches after the working-chamber was one-third full of concrete, unless it happened that the pressure was considerably lowered. If the air-pressure was lowered, was it done purposely or by accident? It seemed rather strange that the caisson should be sinking 6 inches after the filling of the working-chamber was in hand. No data were given with regard to skin-friction, and he thought all data on the subject might be misleading, partly for the reason that Mr. Cruttwell had given—that in some cases the cutting edge overlapped the actual sinking face—and also because unless the caissons went down absolutely plumb there must be a very wide variation in the results. It was said that the total weight on the foundations at the close of the operations was 9,890 tons. Calculating the upward pressure from the compressed air, having regard to the fact that the caisson was 78 feet deep, he found the remaining downward pressure would be about 5,000 tons, which seemed a very large weight to need for sinking. He had compared it with the caissons at Newcastle, where the weight required for sinking after deduction of the upward pressure was only 2,800 tons; the depth was not quite so great, but the caissons were practically double the size. He did not know whether the reason was that there was excessive skin-friction, and he would be glad to have some information on the matter. It would be interesting, too, if a Table could be added showing details of the sinking of the caisson, on similar lines to the Table given in Mr. Matheson's Paper. Also, if there was a geological section it would be of great advantage to have it added. Dr. Harrison had mentioned the peculiar strata that had to be passed through, and a section would therefore be especially interesting. He thought the work of erecting the girders had been carried out in such an admirable manner that it was impossible to criticize it; he did not think it could have been done better. As a matter of comparison it might be interesting to note that whereas by the

Mr. Kirkpatrick. building-out method adopted at Sunderland the quantity of temporary work used was 5·4 cwt. of steel and 3¼ cubic feet of timber per ton of permanent work erected, in the river at Newcastle, where there were also large girders, by building a staging in the river the quantity of temporary work used was 0·7 cwt. of steel and 18·05 cubic feet of timber per ton of permanent work erected. That comparison he made for the purpose of assisting to settle the question of the way large girders should be erected in the future.

With regard to the Paper on the new Clyde bridge, in connection with the Author's remarks on the foundations he would like to give an example of work he had been connected with in the Severn. Cylinders 15½ feet in diameter were sunk to a depth of 30 feet below the water-line, and the specification provided that 6½ tons per square foot should be actually put on the first cylinder as a test load. That meant that a stack of about 1,200 tons of rails was laid upon the cylinder and remained upon it for 6 weeks. The result was a settlement of 1¼ inch on the fine sand. When the load was taken off again, after 6 weeks, careful tests were made, and by the time the whole of the rails had been removed the cylinder had come up again to exactly its level before the load was imposed. It appeared, from what was said in the Paper, that valuable data might have been obtained with regard to the relation between the quantity of CO_2 in the caisson and illness; and if Mr. Matheson had any data, it would be exceedingly interesting if he would plot a curve to see if there was any definite relation between the two. It was noteworthy that the method of lowering the caisson adopted at Glasgow was almost identical with that adopted at the Tyne bridge. It was singular that two such works should be carried out by almost identical methods without either party knowing what the other was doing. He himself had not known the method adopted at Glasgow until that evening, and as the method had not, he believed, been tried before, it was rather interesting to find that it had answered as admirably in Glasgow as in Newcastle.

Mr. Read. Mr. R. J. G. Read thought The Institution was fortunate in having two such important bridges to consider at the same time. The first thing that struck him was that in the two bridges the same kind of truss was used for the main girders; it seemed to show that that particular form was fairly well recognized as the most economical and convenient truss for such girders. With regard to the foundations, the river piers were carried down to about the same depth in each case, 70 feet. In one case they were founded on the sand and gravel, and in the other on rock. In the Glasgow bridge the pressure per square foot was given as 6·12 tons, but it was not

Mr. Read.

given in the case of the Wear bridge, and he would like to ask the Authors what it was, because from the weights given for the piers, spans, and live load, he made it out to be 6·1 tons per square foot. As to skin-friction, in giving the figure of 6 tons per square foot as the maximum pressure on foundations, Mr. Matheson said that if the skin-friction had been taken into account it would have been about 10 per cent. less. That would bring it on the deep piers to about 12 cwt. per square foot, which was a good deal higher than had been mentioned. With regard to the unit load on the concrete, he noted it was limited to 7 tons per square foot, which seemed to him to be very low for good concrete, especially as it was rammed. It made a considerable difference to the strength of concrete in compression whether it was or was not rammed. In one case he had tested some plain concrete pillars against reinforced-concrete pillars, and the former came out the higher. The reinforced pillar withstood up to about 300 tons per square foot, and the plain concrete of the same pattern about 316 tons per square foot. He took some trouble to ascertain the reason, and he came to the conclusion that it was because the ramming could be done more perfectly in the mould when there were no bars to interfere with the work. The weight of concrete was given in one of the Tables as 130 lbs. per cubic foot in the case of the Glasgow bridge, and as 140 lbs. per cubic foot in the case of the Sunderland bridge. That also depended a good deal on whether the concrete was rammed or not. It was very useful to have the unit weights of metal on which to calculate the weight of structures for payment, because if the unit weights were not given contractors were apt to use their own estimates of them. He had had occasion recently to inspect a quantity of steel castings which were to be sent abroad. No definite unit weight was stated on which they were to be calculated, and the contractors charged them at 0·3 lb. per cubic inch, which worked out to 518 lbs. per cubic foot. Mr. Matheson gave 492 lbs. per cubic foot, which was 0·2847 lb. per cubic inch. He knew that 0·3 was too high, and he asked several steelmakers at what figure they took the unit weight of cast steel, and all of them turned to reference-books for an answer. Not one of them had actually weighed a piece of cast steel and found the proper figure from such a test. Even Mr. Kirkaldy had not measured the weight when he asked him about it, but a few days later Mr. Kirkaldy wrote to say that he had cut out some pieces of mild cast steel, which he had polished and weighed accurately, and the weight came out at 0·2823 lb. per cubic inch, which was 487·8 lbs. per cubic foot. So that in

Mr. Read. taking 492 lbs. Mr. Matheson was well over the mark. In the case referred to he himself allowed 490 lbs., which he thought was fair, and it was accepted. An important question was raised in connection with the testing of material, namely, whether it should be tested at the mills or at the works where the bridge was being made. That was a matter requiring definite settlement, because it made a great deal of difference in the cost of inspection. In a case he had had to deal with the contractors maintained that the materials should be tested at the works where the iron was rolled, and he found he had to send inspectors to twenty-two different works. In describing the painting of the Wear bridge, it was stated that the bottom of the bridge was treated with Brigg's solution, and he would like to know exactly what that was. Another interesting point was the adoption of a girder on the Vierendeel system for the small footbridge leading to the signal-box—a girder without diagonal ties. As far as he knew that was the first of the kind used in this country. At the Paris Exhibition some years ago he saw a railway-bridge of about 100 feet span on that system with parallel booms and no diagonals. Finally, with regard to the appearance of the bridges, it was stated by Mr. Matheson that considerable pains were taken to give the bridge a pleasing appearance. So far as the elevation was concerned he thought the attempt had been successful, but he did not think the high piers were any improvement when the bridge was looked at sideways—as it usually was—because it was very seldom that an absolutely true view of the elevation of the bridge was obtained. A view of the pier such as was shown in *Fig. 5* was to his mind not quite satisfactory, inasmuch as it showed a structural pillar cut in half. The large vertical pillars that adorned the Alexander III bridge in Paris were perfect pillars and had not that objection. He thought the bridge over the Wear had a very pleasing appearance; there was no attempt at adornment with unnecessary pilasters or piers. The case in which parallel girders could be used for the small spans and a girder with a curved top boom in the principal span was always satisfactory.

Mr. Boycott. Mr. G. W. M. BOYCOTT remarked that Mr. Matheson had considered the question whether he should have cylinders or caissons and had rightly decided to have caissons, but he did not quite agree with the reasons given in the Paper for that decision. For instance, Mr. Matheson said it would be very uncertain and risky to take cylinders down to a depth of 92 feet below high water. In some work carried out by Mr. Collin at the Barmouth bridge, cylinders were taken down to 96 feet below high water, at a pressure of about 41 lbs. per square inch. Owing to a spring there was a pressure of about 46 lbs. in one of the cylinders. Four cylinders were sunk to 96 feet, and there was one bad case of "bends" in a man who ought not to have gone down, as he had had some warning. He could not help thinking that the fact that inclined sides were adopted for the caissons was one reason why there was so much difficulty in keeping them vertical. He himself had had some experience with large caissons on the Tyne, caissons 113 feet long by 35 feet wide, but there was never any difficulty in keeping them plumb. There were two large girders across the caisson at the bottom, but for the last part of the way they were not used. Although there were no strata which dipped at 1 to 1, there was different material at different points. He thought the difficulties at the Wear bridge were also due to the caissons having inclined instead of vertical sides; and the same thing accounted, he thought, for the excavated material coming in under the cutting edge when the pressure was lowered. At Newcastle they were constantly lowering the pressure, and he did not think they had a cubic yard of excavated material come back. It was said the pressure was always ½ lb. per square inch more than the theoretical figure, but as a general rule the pressure was a little less. With regard to the Clyde bridge, he had scaled the cross girders and found that the height was about 3½ times the width, which he thought was a very dangerous proportion. On the Tyne bridge caissons the height of the girders was twice the width. At Glasgow there were a large number of cases of bends at 22 lbs. and very few at the higher pressure of 30 lbs. per square inch, and Mr. Matheson thought the reason of that was that the men were fatigued. He mentioned in the Appendix that the contractors were required to reduce the period of the shift as the pressure increased, but he did not mention by how much the shift was reduced, and it would be interesting if he would state the length of shift worked at 22 lbs. and the length of shift worked at 30 lbs. per square inch. It would be interesting also to know the number of cases of bends at the Sunderland bridge. Mr. Boycott's experience had been always that the ventilation made no difference to the number of cases, and if, as might possibly have been the case, the air was bad owing to lamps or candles being used it would be rather strong evidence that ventilation was not an important matter. Mr. Matheson stated that the air-locks were provided with valves of a certain size, to prevent the pressures from being raised or lowered too quickly. That was undoubtedly the right thing to do for lowering the Mr. Boycott.

Mr. Boycott. pressure, because it ensured that the pressure was lowered quickly at first; but it was quite wrong for raising the pressure, because the pressure should be raised very slowly at first and the rate gradually increased.

Mr. Gilbert. Mr. **Wm. Gilbert** observed that the erection of the bridge over the River Wear appeared to have needed some considerable amount of special apparatus, and he had intended to ask the Authors if, during the design of the bridge, methods of building out the river span without staging, or other temporary works, were considered; but Dr. Harrison had already dealt with this subject. One arrangement would of course be the cantilever and central-girder system, the finished outline of the bridge taking roughly the shape it had during erection, including the mast and temporary ties. Assuming, however, that the bridge was of the most suitable type for the purpose, it was evident that the erection had been carefully performed, although it appeared possible that one or two details of the arrangements might have given trouble had less care been exercised in carrying out the work. The Authors mentioned that it might have been preferable to have used a pin connection at the foot of the vertical mast in Figs. 14, Plate 4, instead of a wide base with a bolted joint. It appeared to him that this would have been a much better arrangement. For, consider the half girder nearly erected as shown in Fig. 14, Stage 3, and suppose it was desired to lift the projecting end of the main girder. The Authors did this by shortening the inclined ties A or B, using hydraulic rams. Now the top of the mast could not move, otherwise the anchor-ties, C, would be altered in length. The top of the mast being fixed, it followed that the projecting end of the girder could be raised only by altering the angle between the mast and the lower boom of the girder, that was, by opening or closing the joint at the bottom of the mast. The insertion of a pin-joint at the foot of the mast would have made the arrangement more flexible, and the stresses more definite. Putting the mast on the adjacent end of the next girder would also improve matters, as the raising or lowering of the projecting ends of the river span would not then disturb the foot of the mast.

Mr. Matheson. Mr. **Matheson**, in reply to observations in regard to the Clyde bridge, stated that he had not meant to convey that the value of skin-friction should be entirely neglected, but he had suggested that, in considering the safe loads on foundations, the assistance from skin-friction should be considered as merely an additional factor of safety, because it might prove illusory, and it should by no means be ignored in the arrangements for sinking cylinders or caissons. Much depended on the character of the strata. The

Mr. Matheson. cross girders above the working-chambers of the caissons had been designed of a strength to take the full weight of the concrete filling, because the concrete had to be largely deposited before working under compressed air was commenced, and also because it was anticipated that the air-pressure might on occasion have to be reduced, if not entirely removed, and that had actually happened. He thought Mr. Boycott's observations on the proportions of the cross girders were based on a misapprehension. The figures given by Mr. Cruttwell in respect of the cost of the foundations suggested more economical work in constructing the foundations of the Tower bridge; but it had to be borne in mind that the foundations of the Clyde bridge were very much deeper than those of the Tower bridge, and this, together with the fact that the foundations of the Clyde bridge had of necessity to be constructed under compressed air, had necessarily increased the cost of the Clyde bridge work. He maintained, however, that the foundations of the Clyde bridge had been constructed economically, the cost comparing well with the cost of the foundations of other structures carried out under similar circumstances. The lower or preservative layer of the waterproofing was entirely of natural bitumen-rock asphalt, experience having shown the futility of using material of an artificial character. It had been borne in mind that, to use the words of an old Scotch contractor, "Ye canna cheat watter"; and the best materials only had been used in the waterproofing work.

Mr. Buscarlet. Mr. Buscarlet, in reply, referred first to the question of skin-friction in the sinking of the caissons. As had been remarked, it was almost impossible to get that accurately, owing to the fact that both at Glasgow and at Sunderland the working-chamber was banked up practically during the whole time of the sinking. It would, however, be interesting if a Table were added to the Paper for comparison with Mr. Matheson's Table in connection with the sinking.[1] With regard to the unit cost of the substructural work, he found that at Sunderland it approximated closely to what Mr. Matheson himself had given. Excluding the cost of the steelwork of the caisson, Mr. Matheson's cost was on the average £2 15*s*. 6*d*., and at Sunderland it came to about £2 16*s*. 4*d*. or £2 10*s*. 4*d*., according to whether one took into consideration the actual material displaced by the caisson and granite, or the absolute displacement of material by the cutting edge, which of course was greater. Mr. Boycott had remarked that in sinking the caissons on the Tyne no material came into the working-chamber. It would be

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[1] This has been done: see Appendix I, p. 89.—F. C. B.

Mr. Buscarlet. interesting to compare, if possible, the amount of material that came in by various methods of sinking. He found that in sinking the caisson in the Wear the quantity of material actually excavated was in excess of that displaced by the cutting edge of the caisson. This was due not so much to the method of sinking as to the fact that the sinking had to be discontinued for a period of nearly 2 months in consequence of the work in the granite-quarries in Norway being stopped by frost, with the result that the working-chamber partially filled while the pressure was off. Mr. Kirkpatrick had drawn attention to the crushing stress borne by the sandstone from the Locherbriggs quarry being less than that used for the King Edward bridge at Newcastle-on-Tyne from the adjoining quarries. The stone found in these quarries varied very much, and in the Locherbriggs quarry itself some of the material was so hard that even the architects in Dumfries would not use it, while other material was very soft indeed. Therefore the pressure borne by the stone would depend very much on where it was taken from—which quarry, and whereabouts in that quarry. With regard to the name given to the mixture of bitumen and coke breeze, "bitumen concrete," he believed that as used on the Wear bridge it was practically a new material; some better name might perhaps be suggested for it. In sinking the caisson one shaft was used at the east end, and it was found that if the other shaft had been erected it could not have been used during the sinking because the west end of the caisson sank on the whole much quicker than the east, and it had to be banked up at one time beyond the opening of the disused shaft. He did not think there was any real danger in using only one shaft, as the amount of banking left in around the edges prevented the caisson from sinking suddenly. The weight on the caisson during the sinking was not regulated to suit the sinking, but the strakes of shell-plates and granite were built up so as to keep continually at least a few feet above high-water level. It was found that the caisson sank quite fast enough, and that the material could be taken out as quickly as was required. The maximum amount taken out on one day was 179 skips of 8 cubic feet capacity, and the sinking was carried on very quickly throughout. The only thing that retarded it was waiting for the granite, which had to be kept well above high-water level. The type of compressor in use was specially designed for sinking, and had been found so reliable that no duplicate plant was considered necessary. He did not think that the slight taper of the caisson, 5½ inches on each side in a height of 44 feet, would make any difference to the guiding of it during the sinking. There was certainly no difficulty at Sunderland, as, whenever the caisson was out of level, it was easily brought back to its proper position by removing the banking or excavating at the side that required lowering. The great similarity in the sinking at the Clyde and Wear bridges was due to the fact that practically the same plant was used at both places. Considering the great weight on the cutting edge, and the fact that the pressure was reduced during the concreting, it was not, he thought, a matter for surprise that the caisson should have sunk a few inches after the excavation had been stopped. There were only a few cases of "bends," and these were not serious. Perhaps the facts that the pressure was kept higher than was absolutely necessary in some cases and that the ventilation was good might account for this. When he went down he could not detect that the atmosphere was particularly bad. For the lighting of the working-chamber Price's "Palmatine" wax candles were used; the light was not as good, of course, as electric light, but no electric supply was available, and the men did not suffer from the use of candles. "Brigg's solution," which had been used for the painting of the underside of the spans, etc., was a paint composed of bitumen, with the addition of an elastic ingredient and also a very adhesive gum, said to be insoluble in water. The elastic ingredient was added to prevent its drying up and decomposing, and the other ingredient increased its adhesiveness. It had been used for the Forth and Tay bridges and had, he understood, proved to be a particularly good paint for steelwork exposed to highly corrosive influences. Some time before using it for the Wear bridge the undersides of several small bridges on the North Eastern Railway were coated with this paint, and it proved to be most satisfactory. Of course, as with all painting, care must be taken that the surface of the steelwork was perfectly clean and dry before the solution was applied. In his opinion enough attention was frequently not paid to the latter condition, as painting was often done during foggy and damp weather. In answer to Mr. Read's question, neglecting friction, the pressures due to dead and live loads on the foundations of piers 1, 2 and 3, were respectively 5, 8½ and 4½ tons per square foot. Mr. Buscarlet.

Mr. **Adam Hunter**, replying to the discussion, stated that in considering the method of erection of the river span, it was recognized that the deformations of the permanent and temporary work during all stages of the erection of the main girders would cause serious secondary stresses to be developed in the members of the structure, unless special provision was made to keep them within safe limits. The operations which were necessary during the Mr. Hunter.

Mr. Hunter. different stages of the erection caused considerable range and reversals of stress in the members of the permanent and temporary work, with consequent variations in the deformations. The alterations of form and the motions of all important points were carefully computed for all stages of the building and lifting operations, and there was close agreement between the calculated and the actual deformations. The expedients adopted to allow freedom of motion and to reduce the secondary stresses at the various critical points during the erection proved entirely successful, and it was doubtful if, with the knowledge and experience gained, any change in the methods pursued would be advantageous. Mr. Moncrieff had referred to the method adopted to ensure a fairly uniform distribution of the shearing stress over the rivets connecting the short thick ties with the tops of the end posts, on account of the comparatively long length of riveted section. It was evident that, to enable the shearing stress to be distributed uniformly over the rivets, the extension of the metal between the rivet-holes in the end posts would have to correspond exactly with the extension of the metal in the ties. As the relative proportions between the metal under erection-stress in the end posts and the ties differed greatly, it appeared that the rivets in the tie nearest to the gap between the girders would be subjected to a much greater stress than those at the ends. In order to see how far this was true, experiments were made with a strip of india-rubber, corresponding with the tie, which was connected by pegs with a board and subjected to tension in the direction of its length. The elongation of the holes and distortion of the rubber was greatest at the hole nearest to the point of application of the pull, and diminished towards the end of the strip. That experiment could be repeated easily with an india-rubber band pinned to a board. In order to ensure distribution of stress as uniformly as possible over the rivets connecting the ties with the end posts, the method of riveting about one-half of the holes, building out part of the main girders, and then riveting the remainder of the holes, was adopted. The lowering of caissons could be regulated as freely by means of the hydraulic apparatus used at the Clyde and Wear bridges as by means of the screws referred to by Mr. Cruttwell, and the work could be done more expeditiously by the former method. The hydraulic apparatus with the links and cotter arrangement had been used extensively for lowering caissons since the time it was first applied by Sir William Arrol at the Montrose bridge in 1882. The simple comparison of the cost of the piers of the Tower bridge and those of the Clyde and Wear bridges was misleading, on account of the

Mr. Hunter.

large size of the Tower bridge piers compared with those described in the Papers, and the difference in the strata through which the caissons were sunk. It would have been impossible to examine the foundations of the Clyde and Wear bridges without the use of air-pressure, while in the Tower bridge the ground was sealed against the entrance of water after the outer ring of caissons entered the clay. Comparisons of costs were of little value without the full information relating to each case. The comparison of the amount of staging used in the King Edward VII bridge over the Tyne with that in other structures was interesting, but no general rule could be drawn from these figures without information as to the weight and size of each span, the height of the stage, and the number of times it was used. The amount of staging used for the land spans of the Wear bridge per ton of permanent steelwork erected was very small, but it must be remembered that those spans were very heavy, and nearly as much timber would be used for the same span and height of stage if a lighter structure were to be erected upon it. In a Paper on the erection of bridges, by Mr. Seyrig,[1] similar figures were given for the staging used for other bridges, and those figures showed considerable variations, and that many factors entered into the problem. With reference to the depth to which cylinders might be sunk below water in pervious strata, it was interesting to note that in the railway-bridge over the River Barrow in Ireland some of the cylinder foundations were carried down under air-pressure to a depth of 120 feet below high-water level, and by means of a water-ejector worked by the compressed air in the cylinder the air-pressure was kept down to a maximum of 45 lbs. per square inch. That depth was a record for cylinder work under air-pressure. Referring to Mr. Gilbert's suggestions about the temporary towers, the same difficulties due to the broad base would have arisen had the towers been placed on the land spans, as they would have heeled over towards the river owing to the extension of the anchor-ties and the compression of the bottom boom of the land spans and of the towers themselves. As it was, the tops of all the towers moved 5 inches over from the position in which they were orginally set. Those who had been in charge of preparing the scheme of erection of the Wear bridge, and engaged in carrying out the work at the site desired to acknowledge their special indebtedness to Sir William Arrol for his advice and assistance.

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[1] Minutes of Proceedings Inst. C.E., vol. lxiii, p. 151.

Correspondence.

Mr. Brodie. **Mr. J. B. Brodie** observed that the limits fixed by Act of Parliament regarding spans, widths, and headways seemed to have left little choice to the Author in the design of the new Caledonian Railway bridge. Its utilitarian features were well marked; but no great pretensions could be put forward in regard to the æsthetics of the design. With regard to the heavy live and dead loads upon the bridge necessitating large flange-areas for the main girders, on account, as the Author said, of the indifferent proportions in respect of ratio of depth to span there was not much to criticize, the structure being a deck bridge having a series of parallel girders placed at short intervals apart, diagonally braced and stiffened by the floor. Having regard to the statement in the Paper that massive piers were required, and that the gross weights transmitted to the foundations were very great, it would be interesting to know why the Author had preferred a series of cylindrical piers, resting on monolithic foundations, to a continuous solid pier, which would have done away with the cap-sill girders, and would have distributed the load just as well. The solid pier seemed to be less costly, and the argument advanced in support of monolithic foundations, instead of individual piers as originally contemplated, seemed to apply to the continuous pier as against individual cylindrical piers. In the Author's treatment of the question of loads on subaqueous foundations, the sustaining effect of skin-friction had been entirely neglected, and the diminution of the weight of the submerged parts of the pier had been discounted. Although that might have resulted in a greater unit load being permissible on the foundations, it erred on the score of extravagance. It might be pointed out that the foundations of many large railway-bridges had been designed subject to the consideration of the substantial support afforded by those elements, and were doing their work well. The question was doubtless a matter of opinion, and was largely influenced by the financial resources of the company; but as the foundations were among the most important parts of such an undertaking as a large bridge, it was essential that no doubt should exist as to their stability.

Both the new Clyde bridge and the Queen Alexandra bridge were characterized by solidity of construction wherein the best materials had been used and applied in the best manner. Perhaps the most noteworthy feature in the building of both bridges was presented in the erection of the river span of the Queen Alexandra bridge; and the ingenuity displayed in the design of the arrangements for erection, and the bold and skilful tactics of the contractors, were worthy of all praise. **Mr. Brodie.**

Mr. F. Broomfield, while appreciating the unusual difficulties of the erection of the river span of the Queen Alexandra bridge, and the ingenuity and skill shown in overcoming them, was of opinion that a very unsuitable type of girder had been chosen for the bridge. The question of erection, which should have been the governing factor both in determining the most suitable type of girder to use and in the design of the bridge generally, did not appear to have received serious consideration until after the design had been completed. Not only had that resulted in excessive weight of the bridge proper, but, as the only practicable method of erecting the bridge proved to be by transforming it virtually into a cantilever, a large amount of extra steelwork was required for temporary towers and ties, which might have been economically employed as a permanent part of a much lighter cantilever structure. A great saving would have been effected thereby in the weight and cost of the bridge, and the tedious computations of the stresses and deflections for all stages of the partially erected structure might have been largely avoided; while the time required for the completion of the work might have been considerably reduced. The situation appeared to have been eminently suitable for a cantilever type of bridge, and Mr. Broomfield would like to ask the Authors if there was any reason why that type had not been adopted. **Mr. Broomfield**

Mr. W. Dawson agreed with Mr. Matheson's observations that the testing of steel at the steelmakers' works left something to be desired, and that it would be better if the specimens were selected when the material had been delivered at the bridge-building yard. That had been the practice on the London and North Western Railway for the last 25 years or more, and very little inconvenience had arisen in adopting the principle. On the other hand, a large amount of time had been saved by inspectors selecting specimens for testing at one place, instead of having to visit a number of mills where the various sections were rolled. There was also the important consideration that the engineer had thereby greater assurance that proper material was used in the work. With regard to the ties and struts of the web-bracing not being riveted **Mr. Dawson.**

Mr. Dawson. together where they crossed, Mr. Dawson thought that would in time give trouble, owing to corrosion. Some years ago his attention was called to the bottom flanges of some roof-girders over Oxford station, where the rivets were spaced 15 inches apart, and in consequence there were insufficient means of keeping the flange-plates in close contact with the angle-bars. The result was that corrosion set in, ultimately forming a long cake of rust $\frac{5}{8}$ inch thick, forcing the plates apart and shearing off the connecting rivets. He thought it was now generally agreed amongst engineers responsible for the maintenance of iron and steel structures that in order to maintain them with efficiency and economy it was essential that they should be kept free from corrosion, otherwise great expense in renewals or reconstruction became inevitable in the course of time, accompanied, as it so often was, by dislocation of traffic and other inconveniences. If it were admitted that corrosion should be prevented, did it not follow that mild steel was more economical to use than wrought iron, seeing that it was practically the same price per ton, and that the allowable working-stress, broadly speaking, was 6½ tons per square inch for tension and compression in the case of the former, and 5 and 4 tons respectively in the case of the latter? In other words, the use of mild steel effected a saving of more than 25 per cent. in first cost.

Mr. Fergusson. Mr. J. C. **Fergusson**, whilst regarding both the Papers as valuable additions to the Proceedings of The Institution, did not agree with the Author of the Paper on the new Clyde bridge on the subject of the foundations, and particularly in his method of determining the frictional resistance of the caissons. Although Mr. Fergusson's views were directly opposed to those of the eminent engineers of the Caledonian Railway, he desired to submit them with all deference. The engineers had ignored "in calculating the stability of the foundations, the possible support to be derived from surface friction": hence the areas of the foundations were determined on the basis of 6 tons per square foot, on river gravel 43 to 48 feet deep. If the skin-friction of the caissons were a negligible quantity, why was it necessary to sink caisson C 1, for example, 48 feet deep? As the piers were calculated as columns, and were tapered up with a batter of 1 in 100, there could be no object in increasing their length; and was not the coarse sand and gravel lying between 30 and 40 feet below the river as good a foundation as the same material at 48 feet below? As the scour of the river would never be 33 feet, might there not have been a saving in dead weight and cost of say 15 feet of the caisson? The fact that the bridge was required to carry nine lines of railway did not necessitate deeper

Mr. Fergusson. or larger foundations, in proportion, than would have sufficed for one carrying only a double line, over the same span, for the same live load, provided that the foundations were the width of the bridge. With regard to the Author's methods of determining the frictional resistance of the caissons, he would like to make his argument clear by quotations from the Paper (p. 37). "The excavated material was entirely removed from the working-chamber as low as the cutting edge, and the edge was undercut as much as was possible. The men were then withdrawn from the working-chamber, and the air-pressure was reduced, when the caisson ultimately began to sink." Then it was stated that the gross surface friction was equal to the gross weight of the caisson and its loading, minus the gross pressure of the air on the roof of the working-chamber. In Mr. Fergusson's opinion that was not a correct method of determining the frictional resistance of the caissons: it was certainly much more, for this reason:—When these preparations had been made and the compressed air was let out of the working-chamber, the whole of the working-chamber was converted into a suction-chamber, and it then became a matter of sinking the caisson by a vacuum process. The hydrostatic pressure from without the caisson drove before it what little air was left in the working-chamber, leaving a vacuum; and the caisson resting in gravel and water, aided by its own weight, was drawn down. This process for the moment annulled the skin-friction by drawing water down the outer surface of the caisson to the vacuum created in the working-chamber, and displaced water and gravel equal in area to the size of the working-chamber. That there was an enormous suction-effect exerted by this method of operating the caisson was proved by the tremendous displacement and inrush of material from the bottom and sides of the caisson, which filled the working-chamber to the ceiling, 8 feet high. That was a forcible way of breaking up the gravel foundations, but it was not a correct method of testing the measure of frictional resistance of caissons. The main difficulty in founding the caissons at such a great depth was not in the excavation, but in the sinking of them. The engineers foresaw this, and gave them a batter of 1 in 100: they decided to ignore surface support; still they dreaded skin-friction, and provided against it in order to ease the penetration. The duty of a caisson was to carry weight—not to penetrate. The goods-trains in Canada and the United States were often heavier than those in England, yet in places they were carried along foreshores on lines supported by rows of four piles driven 16 feet to 20 feet apart. Where those piles were "driven to a stop" with a 3,000-lb. ram they

Mr. Fergusson. often derived more support from side friction than from their base. That a pointed pile could not be driven in hard sand and gravel deeper than one not pointed afforded further proof that surface friction, through a certain depth, overcame penetration and resisted sinking. From California along the heavily timbered shores of Oregon, Washington, British Columbia and Alaska—a region in which pile-driving was practised largely—one never saw an iron-shod pile, although some of the piles used in the salmon-traps at the mouths of the American rivers were over 100 feet long. Those piles were sawn lengths of trees cut off square at the top and bottom, and driven with the thick end up, which gave a downward taper of about 1 inch in 9 feet. Such piles drove straighter than pointed piles and would carry more weight than a square sawn pile.

Mr. Husband. Mr. Joseph Husband remarked that the general effect of the approaches of the Queen Alexandra bridge suffered somewhat by comparison with the girder spans. He would like to ask if, as an alternative scheme, consideration had been given to the question of carrying the roadway of the approach-viaducts on columns instead of arches, and if so, how that had compared economically with the work as constructed? A system of continuous columns from foundation to railway, with the roadway carried on cross girders between the columns at suitable heights, would have lent itself to adequate bracing, and would have presented a more homogeneous and satisfactory appearance than the upper tier of columns resting on the arches. It would also have obviated the necessity for planting rather heavily loaded columns on the crowns of the arches, thereby imposing local concentrated loads which, under severe impact and vibration, might cause trouble with the arches in the future. He presumed, from the curvature of the approaches, that high speeds over the bridge were not likely to prevail: was any limit of speed prescribed? The Paper did not indicate the manner in which the columns supporting the railway of the northern approach were dealt with in order to avoid the traffic on the road from Camden Street. The 4-foot lattice girders between the columns were presumably intended as bracing with a view to stiffen the structure longitudinally. Would not bracing-bars of the full depth, or nearly so, of the pillars have been more efficient in this respect? Difficulty had been experienced in satisfactorily filling the pockets between the brackets of the shoe of the caisson. The caisson appeared to be of somewhat unusual design. The horizontal deck-plates carried over the pockets no doubt added to the lateral rigidity of the caisson, but caused trouble in ramming in the concrete beneath them. Horizontal angle-braces carried round the

Mr. Husband. caisson over the pockets would apparently have answered equally well, and would have left a freer space for the filling. In this connection he thought a caisson with a double shell was generally preferable, although a little more expensive to construct. It occurred to him that a slight upward slope given to the roof of the working-chamber of a caisson, from the outer walls inwards to the bases of the shafts, would considerably facilitate the final packing of the concrete beneath the roof of the chamber. He would be glad to know whether that had been tried? Coming to the superstructure, he supposed the close spacing (6 feet apart) of the cross girders carrying the roadway had been necessary in order to keep their depth within a reasonable limit, seeing that several of them occurred over the piers. Otherwise it appeared a disadvantage to attach them to the bottom boom at other points than the junctions of the main bracing, and must have entailed an appreciable increase in section of the bottom boom for meeting the consequent secondary bending stresses. Their greater number, as compared with a wider spacing, also increased the labour of connecting them with the longitudinal stringers. He ventured to doubt the efficiency of the type of track and rail-guards adopted. A very slight slackening of the fish-bolt nuts would tend to set up lateral rocking of the rails, and would cause them to bear on the timber waybeams; whilst he was of the opinion that a heavy balk of timber protected by a steel nosing was more suitable as a rail-guard than a relatively light steel section. The method of bolting the temporary towers, used during the erection, to the ends of the river spans appeared to be inferior to the adoption of a hinged tower. He was aware that in England there was a certain degree of reluctance to employ hinged members, but the excellent results obtained on the Continent and in America had fully demonstrated their value. Compared with the erection of the Niagara arch of 840 feet span, in which adjustable anchor-ties were used in connection with pin joints, the bolted bases of these towers and the manœuvring thereby involved seemed a little crude. In an important work such as this it would be of great interest and value if the calculations relating to the deflection could be placed on record. Reliable methods of calculating the exact deflection of even a simple lattice structure were rare; while the calculation of the deflection of a doubly-braced girder, in which the ties and struts were riveted at the intersections, must have been particularly intricate. One other point he wished to comment upon was the position of the bearings on the river piers. According to Fig. 13, Plate 4, the pin-bearing for the river span was 5 feet from the centre of the pier, and the

Mr. Husband. expansion-bearing of the land span about 2 feet 9 inches. The dead load alone on each bearing of the river span was 800 tons, and on each bearing of the land span 371 tons, while still heavier loading would come upon the river-span bearings during erection. Taking the case when the river span was carrying the rolling load and the land span was unloaded, the total pressure on the river-span bearing would be about 1,300 tons, and that on the land-span bearing 371 tons. The resultant of these unequal pressures would be situated about 3 feet 3 inches from the centre of the pier towards the river face, whilst with the land span loaded and river span unloaded the resultant would only move over about 1 foot 9 inches towards the pier centre. Would it not have been preferable to set back the bearings about 2 feet 6 inches farther from the river face of the pier, with a view to obtain a more nearly central loading of the pier? It appeared from the drawings that the joint between the land spans and the river spans had been placed over the centre of the pier without having regard to the large difference in span and loading. This would further have proved advantageous during the early stage of the erection, since an additional length of 2 feet 6 inches would have been available in rear of the front packing-blocks. He did not quite understand the statement that the tests indicated that under ordinary working conditions the bolts or rivets in a joint were not called upon to resist shearing stress. Unless that implied that the whole pull was resisted by friction, he would like to ask in what way the rivets were resisting? In a riveted or bolted joint, where the rivets or bolts were only loaded to a safe extent, one would scarcely expect them to show signs of shear effect or distortion, unless they had been subjected to severe reversals. Assuming a turned and fitted bolt of $1\frac{1}{16}$ inch diameter under a mean working shear of, say, 4 tons per square inch, and a reasonable coefficient of friction, it should not need very heavy hammering to remove it from its hole. The ease with which bolts were knocked out whilst under stress was scarcely a proof of the above statement.

Referring to the new Clyde bridge of the Caledonian Railway, a point in the design to which exception might be taken was the uneconomical ratio of depth to span in the main girders. It was, however, so obviously impossible, on account of the relative levels of the work, to give the girders a greater depth that this could not be regarded as a fault. It was especially satisfactory to note the care that had been taken to secure for the bridge a pleasing appearance. In a bridge of this type, which was so utilitarian in character, small blame might attach to the designers for comparative neglect of this feature. He did not wish it to be

Mr. Husband. thought that he was in the least degree making any invidious comparison, but he would like to direct attention to two outstanding features which were strongly emphasized by these two bridges. Each possessed a large central span, with smaller spans on either side. The two noticeable features were, first, the long horizontal line of the upper edge of the parapet in the Clyde bridge as compared with the interrupted upper edge of the girders of the Wear bridge; and secondly, the projecting piers of the former as compared with the truncated piers in the latter bridge. There were, of course, the soundest of structural reasons for these differences. The upper booms of the river-span girders of the Wear bridge were necessarily curved on the score of economy; yet there was no doubt that the uninterrupted parapet-line of the Clyde bridge largely enhanced its appearance. Again, the carrying-up of the piers of the Clyde bridge considerably above the level of the girders characterized the structure in a very marked manner as contrasted with the Wear bridge. The presence of the overhanging footways and pipe-brackets on the latter bridge necessarily precluded any such treatment of its piers. The contour of the sky-line and the treatment of the piers constituted the two most noticeable and characteristic features in any bridge from the point of view of æsthetics.

Mr. Robertson. Mr. F. E. Robertson thought the Author of the first Paper was probably correct in his remarks to the effect that the natural pressure at the bottom of a foundation did not seem to receive much consideration, although it was indeed the principal element to be considered. For example, 7 tons per square foot on sand would be an absurd pressure to attempt on a shallow foundation, but at 150 feet deep it was no more than a balance to the surrounding pressure, and it was the surcharge only which counted. Rankine's formulas for foundations, which were as good as any that could be devised, depended explicitly upon the ratio of the unit pressure from the structure to that naturally existing. In most accounts of works the facts relating to the pressures on foundations were given in a very perfunctory way, it sometimes not even being stated whether skin-friction and displacement had been deducted. The former, as a rule, was hardly worth doing, as a safe amount would be very small. With reference to the remarks on the effects of compressed air when in the clay, it might be pointed out that the proper place for the fresh-air inlet was in the working-chamber, as far from the shaft as possible: if the air did not escape through the soil it should be deliberately wasted through the lock, so that at no time should there be less air passing than 5 cubic feet

Mr. Robertson. per man per minute at the pressure used, and too much air could not be given. To discharge the air from the compressors just below the lock meant that the fresh air was consumed in locking out while the foul air remained in the working-chamber. There was a Blue Book,[1] containing invaluable information on deep-water diving, which could be purchased for a shilling. The suggestion on p. 41 as to standardizing practice in bridge-loadings and permissible stresses was a timely one; but that was work which should be done by a Committee of The Institution. The Indian Bridge Rules were good and might be adopted with advantage. They took the form of a permissible stress referred to dead load, the figure being 8 tons per square inch. The live load was reduced to terms of dead load by an impact allowance which varied with the length loaded, from 100 per cent. on a zero span down to 60 per cent. on 200 feet span, and so on, so that the working-stress on a stringer, where the ratio of dead load was very small, would come out at about 4 tons on the static load, and the stress in the braces would vary automatically, according to the ratios of dead and live load, producing the maximum stress in the member. For compression-members a column-formula was used based on a maximum stress of 6·8 tons, however small the ratio of L/ρ might be. To standardize the loadings would be a more difficult matter, as circumstances varied so much; but a series, such as Cooper's, might well be drawn up, which should preserve the proper relation of load per foot of track to the length loaded in the case of railway-bridges. The stilt expansion-bearing used in the bridge in question had its merits, but had the fault of producing a positive movement of raising or lowering the end of the girder, which might be small in amount but was there all the time.

The Paper on the Wear bridge was of special interest to Mr. Robertson, as he happened to be engaged upon a very similar problem, the design of 360-foot double-track spans for India, some of which might have to be erected by building out. There were generally two ways and sometimes more of dealing with such a problem, and if he had arrived at different conclusions from those arrived at for the work under consideration, they might form an interesting subject for discussion. As regards the curbs of the caissons, an open frame was better than a horizontal plate at the top, as it was difficult to pack the concrete under the plates. His practice was to put a similar horizontal open frame half-way up the wedge, so that the whole, when concreted up, made a very strong girder. He preferred to make the roof of the chamber slope upwards towards the shafts, so as to render the filling

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[1] Report to the Admiralty on Deep-Water Diving. London, 1907.

Mr. Robertson.

of the chamber easier. Building out was clearly the right way to deal with the river span, but with unimpeachable foundations he would have preferred to use a continuous girder. To strengthen a simple span for building out added about 8 per cent. to its weight, which 8 per cent. did not increase the capacity of the girder for its normal load. But the disposition of the metal in a continuous girder suited both the building out and the permanent loads. The girders, being so heavily loaded, seemed too shallow, and the panels were very short. He would have made them 15 feet and used sub-panels. In building out the sub-panel members could have been omitted until the girders were joined; and saving half the joints in building out was no small matter. It was not clear why the road-way cross girders were placed closer together than those of the railway, as the load on them was much lighter; and to put the lower chord under cross-bending loads was not economical. The end posts were enormous, and to make a compression-member too wide was apt to lead to poor detail. It was a very useful empirical rule that plates in compression should not be thinner than $\frac{1}{30}$, or at least $\frac{1}{40}$ of their unsupported width. Now the plates of this post must be 48 inches wide, and by this reckoning should be about 1¼ inch thick. Their thickness was not given in the Paper, and it would be interesting to know it. Mr. Robertson had used turned bolts in temporary work and in strengthening old bridges where sound rivets could not be driven: he had found them as tight as rivets, but he preferred to make them with a slight taper. It might be pointed out, though it hardly applied to these bridges where the piers were very stout, that two fixed bearings should not be placed on one pier, or one pier had to carry a double longitudinal load while another did nothing. In these days of continuous brakes the longitudinal thrust was worth noticing and so in a large span was the wind-load. It was not generally remembered that the longitudinal wind-load was about three-quarters of the side load; and when the wind blew at 45 degrees it affected a pier more strongly than a similar beam wind. He had planned for building out that the fixed bearing should be under the anchor span and the roller-bearing left free under the cantilever, so that no undue strain should be brought upon the holding-down bolts. The claim that considerable care had been taken to avoid secondary strains could only be admitted in the sense that the erectors took considerable trouble to mitigate the inconvenience caused by some features in the design, notably the enormous base of the pillar. If a knuckle-joint was good enough to carry the whole girder with the pillar on top of it, Mr. Robertson. why was it not good for the pillar itself? Then the temporary tie-plate on top of the end posts, with a kink in it, could hardly be considered a desirable arrangement. The main ties again, starting from the lower-chord gussets and passing round the wider top flange-plates, showed a considerable deviation from the straight. The thrust-block between the bottom of the end posts had a very wide bearing between surfaces, whose inclinations varied with the load. Mr. Robertson provided the bearing of such wedges with a means of swivelling, as a large bearing-area was of no use unless the pressure was fairly distributed over it. Even in the permanent structure, if Fig. 13, Plate 4, correctly represented the bearings, the centre of pressure was about 10 feet from the river edge of the pier, so that 20 feet was all the useful width the pier need have had. He believed that the points raised afforded fair subjects for discussion, without in any way detracting from the merits of a fine work brought to a successful conclusion without accident.

Mr. Stoney. Mr. E. W. Stoney, referring to the erection of the centre span of the Queen Alexandra bridge, observed that owing to its great weight the work had been very difficult, and had been carried out with great skill and accuracy. The comparative tests of joints made with turned steel bolts and with rivets were very instructive, and showed that turned bolts fitting closely in drilled holes might be used with complete confidence, their use for all temporary work being much more convenient than that of rivets. There was the additional advantage that they were easily inserted, and could also be quickly removed without racking or injury to the plates connected, which was apt to occur when temporary rivets had to be cut and drifted out.

With reference to the new Clyde bridge, it was stated that, the caisson being partly sunk into the bed of the river without any excavation having been removed, the working-chamber was usually found to be partly filled with soft slurry, such material was difficult to remove, and had to be bailed out with buckets before excavations proper could be commenced. Mr. Stoney suggested that such material might easily have been removed by any of the forms of dredgers used in India for sinking wells, cylinders, and caisson bridge-foundations, and he would like to know why such a method had not been used. The arrangements made for waterproofing the bridge-floor and for draining the ballast seemed to be complete and thorough. This matter of waterproofing and drainage was very important in relation to the preservation of girder-work from corrosion, and in older bridges it did not seem to have received that attention from designers which its importance deserved.

Mr. Waddell. Mr. J. A. L. **Waddell** confined his remarks to a few salient points of the design of the superstructure of the new Clyde bridge. The trusses were excessively shallow, a feature apparently unavoidable on account of existing conditions. He had observed, though, both when examining English bridges and from perusal of descriptions of them, that the truss-depths used in English practice were far too small for economy of metal. The panel-lengths adopted were too short. For both economy and appearance they should be fully twice as long as in the bridge referred to. There was no good reason for employing the double system of cancellation, or the Linville or Whipple truss, because the single system of cancellation would be more economical and would present a better appearance; besides, there was a certain ambiguity of stress-distribution in the double intersection, which was objectionable. The use of flats instead of shapes for diagonal truss-members was not good practice, because it was impossible to make the components of the pairs or groups of flats act equally. Often some of such components were quite idle under the dead load, and did not become really taut until most of the live load was applied. In the case of sections that were properly attached to each other by lacing or diaphragms, this unequal distribution of stress on the component parts of the member did not exist to any appreciable extent. There were not enough counters in the trusses, and if it were not for the resistance to shear of the great chord sections the structure would certainly collapse. Mr. Waddell could not show absolute proof of the correctness of this statement, because the live and dead loads for which the trusses were designed were not stated in the Paper, but the following would indicate the justness of the claim:

Let w denote the equivalent live load per lineal foot per truss.
w_1 „ the dead load per lineal foot per truss.
i „ the coefficient of impact.
p „ the panel length.
$W = w\,p$ denote panel live load.
$W_1 = w_1 p$ „ panel dead load.

The number of panels in the truss was twenty-four, and they were almost all of the same length, hence they would be assumed to be exactly so. The tenth panel had no counterbracing, and the maximum net shear on it would be given by the expression,

$$S_{10} = \frac{1 + 3 + 5 + 7 + 9}{24} W (1 + i) - W_1$$

$$= \frac{25}{24} W (1 + i) - W_1.$$

Mr. Waddell. As the panel live load W for a span of 200 feet was certainly greater than the panel dead load W_1, the value of the shear, S_{10}, was positive; and it could be resisted only by the sections of the top and the bottom chords. These, not being proportioned for shear, certainly must be overstressed. The shear on the ninth panel was given by the expression,

$$S_9 = \frac{2 + 4 + 6 + 8}{24} W (1 + i) - 1{\cdot}5 W_1$$
$$= \frac{5}{6} W (1 + i) - 1{\cdot}5 W_1.$$

Taking w, w_1, and i from American practice, and assuming that each truss carried a single track, also allowing liberally for the weight of the ballast, there resulted:—

$$W = w p = 6{,}250 \times 8 = 50{,}000 \text{ lbs.}$$
$$W_1 = w_1 p = 5{,}000 \times 8 = 40{,}000 \text{ lbs.}$$
$$i = 0{\cdot}7.$$

$$\therefore S_9 = \frac{5}{6} \times 50{,}000 (1 + 0{\cdot}7) - 1{\cdot}5 \times 40{,}000 = 11{,}000 \text{ lbs., nearly.}$$

That showed that in American practice, if this type of span were adopted (which it certainly would not be), the ninth and tenth panels would be counterbraced. It was true that the value of the coefficient of impact employed—namely, 70 per cent. of the live load—was probably greater than would ever actually occur, for it was intended to contain a margin of safety; nevertheless, in Mr. Waddell's opinion the upper diagonal in the ninth panel should certainly be either stiffened or crossed by a counter. However that might be, no one could deny the necessity for counterbracing the tenth panel. That such omission of necessary counterbracing was not the universal practice in England was shown by the Paper on the Queen Alexandra bridge. In it there was a riveted span of almost exactly the same length (200 feet), in which one-third of the panels were counterbraced, while in the new Clyde bridge only one-sixth of the panels were counterbraced. Such insufficiency of counterbracing reminded Mr. Waddell of an amusing incident which occurred some years ago at a highway-bridge letting in the United States. There, unfortunately, there were few legal restrictions in connection with the building of bridges, and anybody was at liberty to put up any kind of a bridge that he liked—it being understood that the owner of the structure was responsible for all damage to persons and property which might be caused by its failure. Most of the road-bridges in the United States were built

Mr. Waddell. by the counties under the superintendence of commissioners, who were generally ignorant of everything pertaining to bridge-design and construction. It happened, about the time referred to, that a certain irresponsible builder was making a good thing out of a patented lenticular truss bridge, which involved the use of a much smaller amount of metal than the corresponding ordinary highway-bridge. His *modus operandi* was to attend the lettings with a model of the bridge he advocated, about 4 feet long, to support it upon the edges of two chairs, and to invite two of the heaviest men in the room to sit upon it; and when it did not fail under the load thus imposed, he claimed its superiority over all other types of highway bridge, and often secured contracts thereby. On one such occasion there was present a shrewd contractor who noticed there was no counterbracing in the trusses, and when the two heavy men, who were sitting on the model, were told to rise, he exclaimed, "Hold on a minute. Let Mr. A. rise first." Mr. A. did so, the result being that the span collapsed. The box section of the bottom chords in the new Clyde bridge was objectionable, in that it would hold more or less water and would collect dirt, snow, and ice, thus subjecting the metal to rapid deterioration from rust. In approved American bridge-practice, stiff bottom chords were built open both above and below. There was apparently too much metal in the cover-plates of the chords in comparison with the webs and the flange-angles. The lateral bracing of long single 3½-inch by 3½-inch by ⅜-inch angle-bars, with their flat inclination, was too flimsy, and was not commensurate with the magnitude and importance of the parts connected thereby. Moreover, the two rivets at the end of each angle-bar did not make a proper connection, and three was the least number that ought to be employed.

Mr. Wells. Mr. L. FORTESCUE WELLS did not agree with the suggestion made by the Author of the Clyde bridge Paper that all boring contractors should be officially accredited. The Author stated that it was unusual for the initial pressure of the original superincumbent earth to receive much consideration. Undoubtedly it ought to be considered when dealing with deep foundations and it was the practice of some engineers to do so. The Author entered a plea for the standardization of safe loads on foundations, but how was it possible to do this? So much depended upon local circumstances and the personal element. The depth to which it was advisable to excavate in the generality of cases became largely a matter of judgment on the part of the responsible engineer. The fact that recent practice tended towards an increase in the permissible load on various materials was worthy of attention. In earlier works engineers no doubt often erred

Mr. Wells. on the side of safety. The Table on p. 15 afforded an example of an increase in loading over what was formerly the usual practice. Ten tons per square foot used to be a very generally accepted safe working-load on blue brickwork in cement; the Author allowed 15 tons. On granite ashlar masonry the Table gave 25 tons, and on granite bearing-blocks 20 tons per square foot. At the Tower bridge 16 tons per square foot was fixed as the maximum permissible load on granite, and 10 tons per square foot was put upon blue brickwork in cement.[1] The concrete at the Clyde bridge was 5-to-1, which seemed unnecessarily rich, and 7 tons per square foot was put upon it. No mention was made of "plums" in the concrete. Were "plums" not permitted, and if that was the case, what was the objection? The occasions when it was necessary to prohibit "plums" were few, and it was difficult to understand why so many engineers still hesitated to use "plums" in concrete, especially when deposited in mass. Mr. Wells did not appreciate the restriction as to grouting brickwork, referred to in the Paper. Absolutely good brickwork might be built by laying or paving in a course on a sufficient layer of mortar and then grouting it, the process forming the layer of mortar on which to bed the succeeding course. It did not appear that any material advantage was gained by the through courses of blue brickwork in the piers as shown on the section. He wished to ask whether any settlement of the foundation was recorded before it was decided not to impose the test-load provided for in the contract. Also, what was the maximum load placed on a pile of the staging driven 12 feet into the river-bed? How was it intended to clean and paint the inside faces of the flat bars forming the tension-members of the web-bracing? In the case of a large girder within his knowledge, with tension-members of similar design, the space between the flat bars had been afterwards filled in. He thought the expediency of coating the metal with oil before painting depended on the state of the surface. When the metal could be heated and a really clean surface was dipped in oil which was allowed to form a skin, he believed it to be an excellent practice.

Mr. Matheson. Mr. MATHESON, in reply, stated that the piers had been formed of a series of cylindrical pillars in accordance with the requirements of the City Corporation. It was thought that the series of pillars would give the bridge a better appearance than if the piers were continuous and solid, and in this the railway-company fully concurred. The suggestion in regard to the use of wrought iron as against the

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[1] Minutes of Proceedings Inst. C.E., vol. cxxvii, p. 36.

Mr. Matheson. use of steel in bridge-building was that the cost of maintenance would be less were the former material used. It was well known that steel needed much greater attention in cleaning and painting than wrought iron, because steel corroded much more quickly. In regard to surface friction and buoyancy in connection with the foundations, there was such a thing as expediency, and, having regard to experience and true economics, the proper course had been followed in determining the safe loads. The actual depths of the several caissons had been finally fixed as the work proceeded and had been determined by the circumstances. As to the observations by Mr. Waddell, that gentleman having indicated that his criticism was based on inadequate data, it was sufficient to say, in reply, that if the bridge had to be built again it would be designed exactly as it had been designed.

Mr. Buscarlet. Mr. **Buscarlet**, in reply, referring to the general effect of the approaches of the Queen Alexandra bridge, observed that the idea of carrying the roadway of the approach-viaducts on columns instead of arches had never been entertained. Owing to the height of the roadway, especially on the north side of the river, such a scheme would not have been, in his view, in keeping with the massiveness of the railway-approaches and river-piers. It was, however, a matter of opinion. The columns supporting the railway of the northern approach were placed in rows of three, one on either side of the roadway, and the others following the curve on the west side of the railway, with additional ones where required. The railway cross girders were continued over the roadway and supported a flooring similar to that of the railway. With reference to the close spacing of the girders carrying the roadway, it was questionable whether the placing of them at greater distances apart would have diminished appreciably the weight of the whole roadway (cross girders and flooring). Apart from this, the choice of depth had been influenced by the headway required over the river and by the necessity of keeping the railway at as low a level as possible. Also, by placing the hangers between the verticals, a clear space for fixing them was obtained. Referring to Mr. Robertson's remarks, the spacing of the railway cross girders was governed by the distance apart of the verticals. The thickness of the plates of the end posts was ½ inch. Two expansion-bearings (not fixed bearings) were placed, as shown in Fig. 3, Plate 3, on pier No. 2.

Mr. Hunter. Mr. **Hunter**, in reply, stated that the broad base for the temporary towers had been adopted to facilitate their erection and to suit the construction of the end posts of the main girders. The slacking back of some of the nuts on the bolts in the bases, as

Mr. Hunter. described in the Paper, had given all the freedom of a hinged joint without its disadvantages. The methods by which the deflections had been computed and the allowances which had been made for the effect of the riveted connections and cover-plates were described in the Paper. The deflection of the cantilever main girder supported on the pier and by the inclined temporary ties (assuming these two points to be in a level line), were first computed. The thrust along the bottom boom of the main girder due to the angle of the inclined ties was included in the forces producing this deformation. The droop of the main girder at the point of attachment of the temporary ties was then computed and was due to the elongation of the front ties and the anchor-ties, and to the shortening of the towers, the end posts and the land spans. The main girder was then considered as revolving around the pin bearing on the pier, and the total deflection of the main girder was determined for any point in the span. Mr. Husband's interpretation of the reference to the stresses in riveted joints under working conditions was correct. Mr. Hunter was pleased to know that Mr. Robertson's experience with turned bolts in bridgework was in agreement with the conclusions given in the Paper. He did not agree, however, that it was necessary to have a taper on the bolts. To ensure a uniform bearing, if any slip to cause bearing stress occurred, the holes should be rimered parallel, and parallel bolts used which had a hard driving fit. The diameter of the bolts should be larger in the body than in the screwed portion, in order to avoid damage to the threads in driving them home. If tapered bolts were used they should be driven in alternately from opposite sides of the joint to ensure that one cover was not taking more stress than the other cover. In regard to the position of the bearings on the cantilever and fixed piers, it was desirable for erection purposes to have fixed bearings on both cantilever piers, but provision must be made on one of these piers to convert the bearings under both land and river spans into an expansion-bearing immediately the junction was made in the centre of the cantilever span, and for these bearings to remain so until the temporary work was removed. An important point to remember, and one which had been demonstrated in the erection of the Wear bridge, was to see that before beginning the erection of the cantilever span the locked rocker-bearings were set sufficiently over towards the cantilever span to allow for the extension in length of the bottom boom of the main girders when they were relieved from the support of the temporary ties and became an independent span. The temporary tie-plate at the top of the end posts was an additional tie for emergency purposes. The set in the tie-plate came immediately below the base of the towers, so that the objection raised to the set in it was not a serious matter. The long length of the inclined temporary ties rendered the slight set in them required to pass the top booms of little moment. The great strength and stiffness of the end posts of the main girders ensured a fairly uniform distribution of the thrust over the section of the bottom booms of the main girders due to any variation of the centre of pressure on the thrust-blocks. The precaution was taken, however, to stiffen and brace the upper edges of the vertical web-plates of the boom to resist any increased pressure. With light end posts a swivelling arrangement would have been necessary to keep the centre of pressure in one line. In addition to swivelling motion, vertical sliding must also be provided for. Mr. Hunter.

12 April, 1910.

ANTHONY GEORGE LYSTER, Vice-President,
in the Chair.

The discussion upon the Papers by Messrs. Matheson and Buscarlet and Hunter, on the new bridges over the Clyde and the Wear, occupied the evening.

19 April, 1910.

JAMES CHARLES INGLIS, President,
in the Chair.

It was resolved—That Messrs. E. R. Dolby, C. O. Grimshaw, F. Hudleston, S. R. Lowcock, E. W. Monkhouse, B. Mott, T. Frame Thomson and H. Woodall be appointed to act as Scrutineers, in accordance with the By-laws, of the Ballot for the election of the Council for the year 1910–11.

The Council reported that they had recently admitted as

Students.

Harry George Gould Clarke, B.Sc. (Engineering) (*Lond.*)
Thomas Alexander Finch, B.A. (*Dubl.*)
Frank Richard Freeman.
Christopher Rowley Hill Kane.
Charles Bernard Mathews.
Harrison Moore Murphy, B.A. (*Royal*).
Rohintan Nussurvanji Mirza, B.Eng. (*Liverpool*).
William Percy Nevett, B.Sc. (*Birmingham*).
Jasper Prescott Phillimore.
John Graham Robinson, B.A. (*Cantab.*)
Edgar Cecil Ross.
John William Trodd.
George William Williamson.
Marsham Elwin Wright.

The Scrutineers reported that the following candidates had been duly elected as

Members.

Stanley Hunter Jones.
Jonah Walker Smith.

Associate Members.

George Simpson Baxter.
Ralph William Butler, Stud. Inst. C.E.
Charles Erik Capito, Stud. Inst. C.E.
Maximilian Knight Cooper-King.
Roland John Dunderdale, B.A. (*Cantab.*), Stud. Inst. C.E.
Thomas Maxwell Fyshe, B.Sc. (*McGill*).
Reginald Francis Boyd Gaudin, B.Sc. (Engineering) (*Lond.*), Stud. Inst. C.E.
Alfred Guest, B.Eng. (*Liverpool*), Stud. Inst. C.E.
William Edward Hogg.
Geoffrey Richard Kinder, Stud. Inst. C.E.
Arthur Lyndon Lawrence.

Associate Members—continued.

Leopold George Esmond Morse, B.A. (*Cantab.*), Stud. Inst. C.E.
Alexander Roderick Murray.
Horace Watson Nicholson, B.Sc. (Engineering) (*Lond.*)
Richard Hutton Osler, Stud. Inst. C.E.
Joseph Arthur Edouard Desbarrières Pougnet.
Harold George Rowley, Stud. Inst. C.E.
Frank James Salberg, Stud. Inst. C.E.
Henry Ernest Theodore Vogel.
Francis Henry Walker.
Charles Gordon Woolway, Stud. Inst. C.E.

(*Paper No. 3834.*)

"The Use of Reinforced Concrete on the Wabash Railroad, U.S.A."

By Ernest Romney Matthews, Assoc. M. Inst. C.E., and Andrew Oswald Cunningham, M. Am. Soc. C.E.

Railway-engineers in the United States were among the first in that country to recognize the advantages of using reinforced concrete in engineering construction, and this material has now been used extensively on the railways there, more particularly in the building of bridges, culverts, subways and retaining-walls.

The Wabash Railroad Company was one of the pioneers in such construction, which it commenced to use as early as 1902. This railway is one of the leading lines of the middle west, and has a total length of 2,500 miles. Its headquarters are at St. Louis, and it extends to Chicago, Detroit, Toledo, and Buffalo on the east of the Mississippi River, and to Kansas City, Des Moines, and Omaha on the west.

This Paper gives particulars of the use of reinforced concrete in various engineering works which have been carried out during the past 7 years on the Wabash Railroad, and also indicates the nature of the calculations upon which the design of the various structures was based.

Design.

In designing the structures, the following dead loads were assumed :—

Earth.	100 lbs. per cubic foot.
Concrete	140 „ „ „ „
Rails, sleepers and fastenings	For each track 400 lbs. per lineal foot.

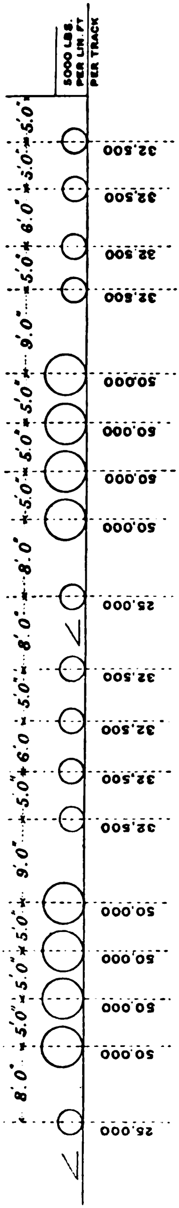
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Fig. 1.

STANDARD LOADING.

(*The loads given are in pounds per axle.*)

The live load was assumed to be equivalent to two coupled "Consolidation" engines and tenders, followed by a uniform train-load, placed so as to give the maximum stress in the structure, as shown in *Fig. 1.*

On account of the sudden application of the live load, additional stresses due to impact are provided for in proportioning the structures, and are determined by the formula

$$I = \frac{L^2}{D + L},$$

in which I is the impact stress, L the calculated maximum live-load stress, and D the calculated dead-load stress. The working-stresses are obtained by combining the maximum dead-load, live-load and impact stresses, and are not allowed to exceed the following amounts:—

	Lbs. per Sq. In.
Steel, tension on net section of rod	18,000
Steel bond on deformed bars	100
Concrete, compression in cross-bending . . .	800
Concrete, direct compression	600
Concrete, shear (diagonal tension) in plain concrete	30
Concrete, shear (diagonal tension) when web is properly reinforced . .	100

These allowable unit stresses are based on the following specifications for steel and concrete: Steel bars to have an elastic limit of 55,000 to 65,000 lbs. per square inch; concrete to be made of 1 part of Portland cement, 2 of sand and 4 of stone or gravel.

The formulas used and the method of calculating the stresses are explained in Appendix I.

Reinforcement.—Many American engineers do not look with favour upon the practice of using plain bars as reinforcement in concrete. Their contention is, and the Authors agree with it, that proper adhesion of the concrete cannot take place unless the bars are either provided with some form of ribbing device, such as the Johnson corrugated bar affords, or twisted as in the Ransome bar, or rolled in the shape of the Thacher bar, or formed in some similar manner.

Most of the reinforcement used in the structures referred to in this Paper has consisted of corrugated bars, in the form invented by Mr. A. L. Johnson, M. Am. Soc. C.E. These bars are square in section, with corrugations on all four sides, and are made in various sizes. The following Table shows the net sectional area and weight of certain sizes:—

Sizes in Inches.	Net Section. Square Inches.	Weight. Lbs. per Foot.
½	0·19	0·78
¾	0·38	1·56
⅞	0·55	2·25
1	0·70	2·90
1¼	1·10	4·56

The sides of the ribs are practically at right angles to the axis of the bar. The bars are rolled from high-carbon steel, with a breaking stress of about 105,000 lbs. per square inch, and an elastic limit of 66,600 lbs. per square inch.

Concrete.—The concrete used in these works was what might be termed a wet mixture, it being the usual practice in the United States in reinforced-concrete work to put in the concrete in this manner. The advantages of so doing are, first, that a more dense and homogeneous concrete is obtained, and secondly, that the reinforcement becomes better embedded, every portion of it being well covered by the cement; rusting of the metal is thus prevented.

It was thought at one time that an excess of water might cause a serious reduction in the strength of the concrete, but such a fear has been found to be to a large extent groundless. Very complete tests were carried out in this connection by Mr. G. W. Rafter, M. Am. Soc. C.E., in 1896, for the State Engineer's Office of New York.[1] Mr. Rafter endeavoured to ascertain the relative strength of 12-inch cubes of dry, plastic, and wet mixtures. These blocks were 18 to 24 months old when tested, and the average compressive strengths were as follows:—

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[1] A. W. Buel and C. S. Hill, "Reinforced-Concrete Construction," p. 359. London and New York, 1906.

	Number of Blocks tested.	Average Compressive Strength.
		Lbs. per Sq. In.
Dry mixture	156	2,470
Plastic mixture	144	2,294
Wet mixture	148	2,180

BRIDGES AND CULVERTS.

The Sangamon River Bridge (Figs. 2 and 2a, Plate 5).—This is a double-track, four-arched skew bridge over the Sangamon River. The abutments and arches are of reinforced concrete, and the piers are of solid concrete reinforced at the base with 1-inch square rods set 4½ inches apart[1] transversely. The toes are also reinforced with five 1-inch square rods placed longitudinally. The piers rest on pile foundations, 259 round timber piles carrying each pier; these are driven into very stiff clay, and are spaced 3 feet apart from centre to centre longitudinally, and 2 feet 6 inches transversely.

The angle of skew is 45°; each arch has a semicircular square span of 61 feet in the clear and a skew span of 100 feet between the centres of the piers.

The arch-rings, which are 3 feet 9 inches in thickness at the crown, are heavily reinforced with upper and lower ribs of 1-inch square rods 12 inches apart, crossed by 1-inch square rods about 4 feet apart, laid parallel with the piers. The joints in the ribs are formed by overlapping the rods. The minimum cover over the reinforcement is 2 inches, and the thickness of the concrete below the ribs in the arches ranges from 6 inches at the crown to 24 inches at the springing.

The features of particular interest in this bridge are the abutments and the method of providing square bearings for the arch-ribs at the skew-backs. The abutments (Fig. 2a, Plate 5) are practically boxes divided by horizontal and vertical partitions into four compartments. The side walls, the partitions, and the slab for carrying the road-bed are all 3 feet thick. The side walls are reinforced near their outer faces with vertical rods 24 inches and horizontal rods 36 inches apart. The slab for carrying the road-bed is reinforced with longitudinal rods 24 inches and transverse rods 4 inches apart, the ends of alternate rods being bent up at an angle of 45° for a length of 3 feet 6 inches. The lower slab is reinforced with longitudinal rods 24 inches apart, and transverse rods, all straight, 12 inches apart. The end walls are reinforced with vertical rods 24 inches and horizontal rods 36 inches apart, and with six sets of upper and lower crossed bracing-rods spaced 6 inches apart in the thickness of the wall. Fillets reinforced with bars 12 inches apart are formed at the corners between the slabs and the walls. The parapet and spandrel-walls are reinforced with horizontal bars 24 inches apart placed near the face.

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[1] Throughout this Paper the distance apart of reinforcing bars denotes the distance between centres

The wing-walls for the bridge are reinforced with horizontal bars 24 inches apart. These wing-walls are provided in order to prevent scouring of the toe of the bank during floods, and to keep the bank from sliding. The height from the base of the rail to the bottom of the pier-foundation is 80 feet, and the height of the earthen filling at each end of the bridge is 60 feet.

The cost of this bridge was $123,200 (£25,100)..

Bridge No. 11, Forest Park, St. Louis (Figs. 3, Plate 5).—This bridge consists of concrete abutments carrying an 80-foot double-track, through-girder bridge. The abutments are of the hollow type, namely, thin reinforced face-walls supported by counterforts resting on slab foundations. The principal features are the curved wing-walls, the concrete fascia and balustrade carried by the outside girders, and the other ornamental concrete work. It will be noticed that the counterforts not only carry the girders, but also act as ties to hold the face-wall of the abutments. This face-wall is 2 feet 8 inches thick at the bottom, and is battered in front so that the top thickness is 18 inches. It is 18 feet high and is surmounted by a slab 6 feet wide and 18 inches thick, which serves as a bridge-seat, and rests on top of the counterforts. On the back of this slab a cross wall is built. The construction of the wing-walls is somewhat similar, though the top slab is omitted as shown. The widths of the counterforts are 3 feet for the abutments and 18 inches for the wings, and the wall is thickened at its junction with the wings.

The reinforcement in the abutments consists of ¾-inch square corrugated rods placed at the following distances apart:—

For the face-walls, vertical rods, 24 inches; horizontal rods, 6, 12 and 18 inches, at different levels.

For foundation-slabs—

	Distance apart. Inches.
Bottom longitudinal rods under face-wall below girders	12
,, ,, ,, under middle counterfort	2
,, ,, ,, under end counterforts	4
,, ,, ,, elsewhere	24
Bottom transverse rods under face-wall	4
,, ,, ,, elsewhere	24
Upper longitudinal rods under face-wall	12
,, ,, ,, elsewhere	4
Upper transverse rods	4

In the bridge-seat, the longitudinal rods are 12 inches, and the transverse 24 inches apart; and for the counterforts, the horizontal rods, 6 feet long, are 12 inches apart, and the vertical rods 6, 12 and 18 inches, at different levels.

The tie-rods in the back of the middle counterfort are eighteen in number, placed 6 inches apart. The end counterforts have nine tie-rods. The reinforcement in the wing-walls is similar to that in the abutments except that the rods in the foundation-slabs are all spaced 12 inches apart.

The bridge-floor is of concrete 8 inches thick, and is reinforced in both faces with ½-inch corrugated bars. This slab is carried upon 15-inch steel I-beams, which are riveted by connecting angles to the webs of the girders, and are spaced 18 inches apart between centres. In each abutment 28,700 lbs. of ¾-inch corrugated rods were used.

Reinforced concrete is sometimes objected to on the ground that ornamental structures cannot be built of this material; but the Authors venture to claim that this bridge contradicts such a statement. It is built at the main entrance to the largest and most beautiful park in St. Louis: hence the necessity for æsthetic treatment.

The substructure and the ornamental work were constructed by contract, but the railway-company erected the steelwork themselves. The cost was $22,307 (£4,565), but this included the removal of an old bridge and its stone substructure; also a large quantity of excavation on account of increasing the width of the opening, and the necessary temporary bridging to provide for the traffic. As this bridge was built during the International Exhibition at St. Louis, and as more than three hundred trains crossed it daily, it was impossible to remove the extra earth and the old bridge with trains, and it was necessary to do this with teams, at heavy additional expense.

Reinforced-Concrete Bridge-Floors.—Alternative arrangements of solid reinforced-concrete floors for various types of bridges are shown in Figs. 4–7, Plate 5. The railway-company have proved these to be very successful, and they use them to the exclusion of all other designs on their bridges. The cost varies between 16*s.* and 20*s.* per lineal foot of floor, 13 feet wide.

Where it is necessary to have a watertight floor a mixture of 1 part of cement and 3 parts of coarse sand is used, but where watertightness is not necessary, e.g., where the bridge crosses streams, the slab is made of the normal 1 : 2 : 4 concrete.

In Fig. 4 the reinforcement consists of ½-inch steel bars, spaced 6 inches apart longitudinally in the bottom face of the slab, and 12 inches apart in the top. Transversely the slab is reinforced with ½-inch rods placed 18 inches apart.

In the design shown in Figs. 5 the reinforcement is arranged in a rather different manner. Wire netting, of 1-inch mesh, acts as the reinforcement over the 15-inch girders. This design is used in places where a shallow floor is required.

In Figs. 6 is shown the method of carrying reinforced-concrete floors on a deck bridge. This style of floor for deck bridges, however, has been superseded by one in which the I-beams are omitted and a reinforced-concrete slab 12 inches thick is used in their place, as shown in Figs. 7.

Bridge No. 1383.—This bridge consists of two reinforced-concrete abutments, carrying an 85-foot deck-girder bridge with a concrete slab of the type shown in Figs. 6. The abutments, which are 32 feet high, consist of reinforced-concrete walls supported by counterforts, and are built for double tracks. The buttresses rest on concrete foundation-slabs 15 feet wide by 3 feet thick, which are reinforced top and bottom.

Bridge No. 799.—This is a reinforced-concrete trestle-bridge (Figs. 8, Plate 5). The two abutments are of plain concrete, and the two piers are of reinforced concrete; three reinforced-concrete slabs rest on the abutments and piers. The piers extend a trifle deeper than the abutments. The abutments are 6 feet 6 inches in width at the base and 3 feet at the top, each containing 155 cubic yards of concrete.

The depth of the foundations for the piers is 3 feet; these foundations are not reinforced. The piers are 3 feet 3 inches in thickness at the base and 2 feet at the top, and are reinforced in each face by ¾-inch horizontal and vertical rods 24 inches apart, and at the top by ¾-inch horizontal rods, the longitudinal ones being 5 inches and the transverse ones 24 inches apart. The piers have a batter of ⅓ inch to the foot, and each contains 62·37 cubic yards of concrete.

The slab is 2 feet 6 inches in depth. The lower reinforcement consists of longitudinal ¾-inch square rods 3 inches apart—a length of 2 feet 9 inches at the ends of alternate rods being bent up at an angle rather steeper than 45°—and of transverse rods ½ inch square 12 inches apart. The upper reinforcement consists of ½-inch rods spaced 12 inches apart in both directions. Each slab contains 37 cubic yards of concrete.

The coping is reinforced by ⅓-inch rods spaced 8 inches apart longitudinally and 12 inches apart transversely.

This bridge is for a double track, and the approximate cost was $4,000 (£815).

Bridge No. 640.—This bridge consists of two unreinforced-concrete abutments supporting I-beams, which are encased, and also support a reinforced slab (Fig. 9, Plate 5). The slab is solid, and the ballast is placed on top. The height of the abutments is approximately 11 feet, and the span is 16 feet.

The foundations of the abutments are 5 feet in width by 2 feet in depth, the abutment itself being 3 feet 6 inches in width at the base and 3 feet 3 inches at the top. In each abutment there are 61·5 cubic yards of concrete. The slab is 9 inches in thickness, and the reinforcement consists of longitudinal rods and cross rods, all spaced 6 inches apart. The I-beams are from old bridge stock, and longitudinal rods are placed along the lower flange as shown, sixteen rods to each beam under the rails.

The cost of this bridge was $1,300 (£265).

Bridge over Junction Avenue, Detroit.—This bridge was erected about 2 years ago over an important street in Detroit, Michigan, its chief feature being the method of waterproofing. It consists of a solid-floor plate-girder bridge. It was necessary to make the bridge absolutely watertight, and the waterproofing on the top of the main floor was specified to consist of a 1-inch coating of Wunner bitumen emulsion, cement, and sand. This was not done, however, the bitumen emulsion being mixed directly in the main body of the concrete, making this particular work rather expensive, but satisfactory, as no leaks have occurred since the bridge was erected. The floor used was of the type shown in Figs. 6, Plate 5.

The steel plate-girders are each 72 inches deep and 61 feet 7 inches in length over all, and between them are 12-inch steel I-girders, on which the floor-system is carried. The floor-beams are surrounded by concrete, which is reinforced with ½-inch square longitudinal and transverse rods. The upper surface of the concrete is covered with a 1-to-3 bitumen mortar, which consists of 2 lbs. of bitumen emulsion to every sack of cement (90 lbs.) and 300 lbs. of sand, The cost of this floor was $13.33 (£2 14*s.* 5*d.*) per lineal foot.

Standard Reinforced-Concrete Box Culverts.—These culverts are used for comparatively small openings, being designed for spans of 4, 5, 6, 8, 10, 15, and 20 feet, respectively.

One of the 10-foot culverts is shown in Figs. 10, Plate 5. The bottom slab is 1 foot 6 inches thick, the top slab is 1 foot 8 inches with a slight gradient from the centre each way, and the walls are 2 feet thick. The reinforcement in the bottom slab consists of horizontal longitudinal bars 24 inches apart, and transverse bars (i.e., parallel to the tracks) 6 inches apart; diagonal bars are inserted at the junction with the walls. The walls are reinforced by horizontal and vertical bars 24 inches apart, and the top slab in a similar manner to the bottom one, except that the transverse bars are placed on the lower and not the upper side, and that the ends of alternate bars are bent up at an angle of 45° for a length of 1 foot 9 inches. All reinforcement has at least a 2-inch covering of concrete, and the bars used are $\frac{7}{8}$-inch square; the cement had to pass the standard test adopted by the American Railway Engineering and Maintenance of Way Association in its "Manual of Recommended Practice" of 1905. In a box culvert of this size 2·944 cubic yards of reinforced concrete is inserted per lineal foot of barrel, and 50·8 cubic yards in the two caps, two aprons, and four wings.

For the smaller culverts a similar design is adopted, but the thickness of the concrete is reduced, and $\frac{3}{4}$-inch square bars are used; further, the distance between the transverse bars in the floor and roof is increased (by 50 per cent. in the case of the 4-foot culvert), and the bars at the corners are omitted. In the larger culverts 1-inch square bars are used. The spacing of the reinforcement of the 20-foot culvert (Figs. 11, Plate 5) is practically the same as that of the 10-foot culvert, except that the vertical bars in the walls are placed 12 inches apart. In the 15-foot culvert the spaces between the transverse bars in the floor and roof are increased by 25 per cent. The following is the estimated cost of the various sizes:—

Size in Feet.	Cost per Lineal Foot of Barrel.				Cost of Wings, Caps and Aprons.			
	$	£	s.	d.	$	£	s.	d.
4 × 3	7·50	1	10	6	60·00	12	5	0
5 × 4	9·50	1	18	9	104·50	21	7	6
6 × 5	12·00	2	10	0	132·50	27	0	0
8 × 7	20·50	4	3	6	257·00	52	10	0
10 × 8	25·50	5	4	0	380·00	77	15	0
15 × 10	48·50	9	18	0	562·00	114	10	0
20 × 10	77·00	15	14	6	657·00	134	0	0

Culvert No. 24.—This is a 20-foot by 10-foot box culvert of the foregoing type (Figs. 11, Plate 5), built for a double track. The wing-walls are built slightly on the splay, and slope down at an angle of 45° in elevation. A section of one is given in the Figure. They are reinforced with ¾-inch horizontal bars 24 inches apart, and 1¼-inch vertical bars 12 inches apart. The width of these walls at the base varies from 2 feet 6 inches to 5 feet 6 inches, and the base is reinforced with ¾-inch rods 6 inches apart; the walls are battered at front and back 1 inch to 1 foot, and are surmounted by a reinforced-concrete coping.

Reinforced-Concrete Pipes.—These are made in 3-foot sections with a diameter ranging from 2 feet to 4 feet, and the 4-foot type is illustrated in Figs. 12, Plate 5. The reinforcement consists of woven-wire fencing. In the construction of the pipes, inside and outside forms are used, the casing being 2 inches in thickness, and lined with galvanized iron of No. 20 gauge. The 4-foot pipes are 4 inches in thickness, the inside and outside reinforcement having a 1-inch covering of concrete; and ¼-inch corrugated steel bars running longitudinally are placed 15 inches apart. The wire fencing consists of two strands of steel wire of No. 12½ gauge, the cross wires being of No. 14 gauge; the mesh is diamond-shaped, 4 inches across and 8 inches long. The quantity of material required to make a 4-foot pipe 3 feet in length is:—

Portland cement	1 barrel (= 376 lbs. or 3·8 cubic feet).
Sand	0·3 cubic yard.
Stone or gravel (capable of passing through ½-inch ring)	0·61 „ „
Wire fencing (34 inches wide)	28 lineal feet.
¼-inch corrugated steel bars (3 feet long) . .	22 „ „

and the approximate weight of this size is 2,600 lbs.

The smaller pipes are constructed in a similar manner, the longitudinal rods being spaced somewhat farther apart.

The cost for a 3-foot section is approximately:—

2 feet in diameter	\$3·30 (13*s.* 6*d.*)
3 „ „ „	\$4·50 (18*s.* 4*d.*)
4 „ „ „	\$5·80 (23*s.* 8*d.*)

and the form is estimated to cost \$20.00 (£4 1*s.* 8*d.*).

It was formerly the custom to use pipes 8 feet in length, but in order to make them of this length it was found necessary to use derricks in handling the forms, and platforms to do the pouring from. Further, in shipping 8-foot lengths it was necessary to pay particular attention to blocking, so that the pipes would not roll about and break themselves to pieces. For these reasons it was decided to build a pipe of such length that the forms could be handled by two men and could be filled from the ground without the use of platforms. It is still found necessary to load the pipes by means of derricks, but they can be shipped on end and unloaded and placed in position without machinery.

GENERAL.

No special test loads are used to test a structure after it is completed, as this is considered unnecessary. Inspections are made on all bridges on the Wabash Railway, including, of course, reinforced-concrete structures, by a competent inspector once every month. The engineers in charge of the divisions on which the bridges are situated also inspect them each half-year.

All the reinforced-concrete structures—and there are a great many on this railway—have needed no repairs since they were built. They are very rigid under loads, and their appearance indicates that they will outlast any other kind of structure and require no maintenance. A great many have now been in the track more than 5 years, carrying loads very nearly as heavy as the live load used in designing them. Attempts have been made to determine the deflection in reinforced-concrete structures due to train-loads, but none can be detected by ordinary measurements.

Sufficient time has now elapsed to demonstrate the fact that reinforced concrete is reliable; it is undoubtedly permanent, and as good, if not better, than cut stone for the construction of bridges, culverts, and similar work; it is certainly cheaper. Compared with steel, reinforced concrete is altogether superior, as it requires no maintenance. It is thus in most cases preferable to other methods of construction, and is used where possible on the Wabash Railroad.

The foregoing works have been designed and carried out by Mr. A. O. Cunningham, Chief Engineer to the Company.

The Paper is accompanied by twenty-eight illustrations, from which Plate 5 and the Figure in the text have been prepared; there are also the following Appendixes.

[APPENDIXES.

APPENDIXES.

APPENDIX I.

CALCULATIONS AND FORMULAS.

In proportioning the requisite amount of concrete and steel in the structures, formulas for any load up to the ultimate, based on a parabolic variation of compression and neglecting the tension in the concrete, were used. These formulas are derived and explained by Professor A. N. Talbot,[1] and will be gone into somewhat in detail, so that their use will be understood.

Let n denote the ratio of the moduli of the steel and the concrete (assumed to equal 15).
,, q ,, ,, ratio of deformation existing in most remote fibre to ultimate crushing deformation.
,, d ,, ,, distance from the compressive face to the centre of the metal reinforcement.
,, b ,, ,, breadth of rectangular beam.
,, p ,, ,, ratio of area of metal reinforcement to area of concrete above centre of reinforcement.
,, a ,, ,, area of cross section of metal reinforcement.
,, f ,, ,, tensile stress per unit of area in metal reinforcement.
,, c ,, ,, compressive stress per unit of area in most remote fibre of concrete.
,, k ,, ,, ratio of distance between compression face and neutral axis to distance d.

Then
$$k = \sqrt{\frac{2\,p\,n}{1 - 1/3\,q} + \frac{p^2\,n^2}{(1 - 1/3\,q)^2}} - \frac{p\,n}{1 - 1/3\,q}.$$

At one-fourth of the ultimate deformation, which is about the working load, $q = 1/4$; and when the metal reinforcement is assumed to be 1 per cent. of the total area, and n is 15, then $k = 0{\cdot}43$.

If z denotes the distance from the compression face to the centre of gravity of the compression stresses, then when the concrete at the remote fibre is stressed three-fourths of its ultimate deformation ($q = 3/4$), then $z = 0{\cdot}36\,k\,d$; when $q = 1/2$, then $z = 0{\cdot}35\,k\,d$; when $q = 1/4$, then $z = 0{\cdot}34\,k\,d$; and when $q = 0$, then $z = 1/3\,k\,d$.

When the tensile stresses in the concrete are neglected, the value of the resisting moment of the beam is expressed by the formula—

$$\mathrm{M} = a\,f\,(d - z) = 153\,b\,d^2,$$

where $a = 0{\cdot}01\,b\,d$, $f = 18{,}000$ lbs., and $(d - z) = 0{\cdot}853\,d$.

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[1] University Bulletin No. 14, Illinois, U.S.A. See also Marsh and Dunn, "Reinforced Concrete," pp. 393-401. London, 1906.

The compressive stress in the concrete, due to the foregoing stress in the steel, can be determined by the formula—

$$c = \frac{2af}{kbd} \cdot \frac{1 - 1/2q}{1 - 1/3q} = \frac{2pf}{k} \cdot \frac{21}{22} = 800.$$

This method is the most convenient for calculating the resisting moment, but it depends entirely on the tension of the steel. If the percentage of steel is less than 1 per cent. and the concrete is a 1 : 2 : 4 mixture, then the full compressive strength of the concrete will not be developed.

If it is desired, however, to express the resisting moment in terms of the compression in the concrete, the following equation should be used—

$$M = \frac{(1 - 1/3q)}{(1 - 1/2q)} 1/2\, c\, k\, d\, (d - z).$$

If q is taken as 1/4 of the ultimate deformation, as assumed previously, then the equation becomes—

$$M = 0 \cdot 192\, c\, b\, d^2.$$

Assuming $c = 800$ lbs., and reducing,

$M = 153\, b\, d^2$, which is the same as before.

Bond and Shear Reinforcement.—The amount of resistance to slip in the concrete developed by the rods, or "bond," is measured in terms of the area of the surfaces of the bars in contact with the concrete. Thus it is seen that bond may be compared to the rivets connecting the flange with the web in a riveted steel girder.

Let u denote the bond stress per unit area on the surfaces of the reinforcing bars.
" V " " total vertical shear at the given section.
" m " " number of reinforcing bars.
" o " " circumference of the periphery of one reinforcing bar.

Then

$$u = \frac{V}{m\, o\, (d - z)} = \frac{V}{0 \cdot 853\, d\, m\, o}.$$

At any point in a beam the vertical unit shearing stress is equal to the horizontal unit shearing stress (v) there developed, and this can be expressed by the formula $v = \frac{V}{0 \cdot 853\, b\, d}$.

Besides the horizontal and vertical shearing stresses already noted, tensile, compressive and shearing stresses exist in every diagonal direction. When there is no steel web-reinforcement, all the diagonal stresses are taken by the concrete. If t is the diagonal-tension unit stress, and if there is no tension in the concrete, then $t = v = \frac{V}{0 \cdot 853\, b\, d}$.

If it is assumed that the diagonal tension in a beam is 100 lbs. per square inch, then the necessary reinforcing bars to take care of this stress will be 100 − 30 = 70 lbs. per square inch, since 30 lbs. is the allowable diagonal shear in plain concrete. Therefore, the necessary steel area (a_1) to take care of this diagonal tension can be expressed by the formula—

$$a_1 = \frac{70\, b\, d}{18,000}.$$

Reinforcing bars of this area must be placed at an angle of 45° with the longitudinal axis of the beam, and at distances apart less than the depth of the beam.

APPENDIX II.

SUMMARY OF BUILDING REGULATIONS GOVERNING THE USE OF REINFORCED CONCRETE IN THE U.S.A.

	New York.	Buffalo. (1906 Regulations.)	San Francisco.	Cleveland.
Stress limit on concrete in compression (in beams)	500	500	500	500
Do. (in simple compression)	350	..	..	..
Stress limit on concrete in shear	50	50	75	50
Tensile stress limit in steel	16,000	16,000	1/3 of elastic limit	16,000
Compressive stress limit in steel	..	..	..	..
Shearing stress limit in steel	10,000	10,000	10,000	10,000
Ratio of modulus of elasticity of steel to concrete	12	12	15	15
Bending moment in slabs continuous	$\frac{1}{10} w l$	$\frac{1}{10} w l$	$\frac{1}{12} w l$	$\frac{1}{10} w l$
Bending moment in beam continuous	$\frac{1}{8} w l$	$\frac{1}{8} w l$	$\frac{1}{8} w l$	$\frac{1}{10} w l$
Bending moment in square floor panels	$\frac{1}{20} w l$	$\frac{1}{20} w l$	$\frac{1}{20} w l$	$\frac{1}{15} w l$
Columns—maximum ratio of height to width	12	16	15	16

Note :—w is the total load, uniformly distributed.
l ,, ,, length of beam.

Discussion.

The President. The **President** moved a vote of thanks to the Authors for their Paper.

Mr. Matthews. Mr. **Matthews** exhibited several lantern-slides of the works described in the Paper.

Mr. Elliott-Cooper. Mr. R. **Elliott-Cooper**, Vice-President, wished to call attention to one or two points of the Paper, because he thought a great deal was now being said and done in connection with reinforced concrete which the circumstances did not justify. The object of reinforcing concrete was to obtain increased strength by an entirely different process from the ordinary use of steel. In such work as was described in the Paper, which to his mind would be more correctly described as reinforcing the steelwork rather than reinforcing the concrete, he did not quite see the advantages of the process. Before referring to the structures described, he would like to say a few words upon the subject of why such high results, which appeared to be out of all proportion to the materials used, were obtained with reinforced concrete. He believed that if ordinary concrete without reinforcement were made with the same extreme care in methods and in the choice of materials as was used in the case of reinforced concrete, much higher results would be secured than were obtained at present. People were apt to compare a material, and a form of putting that material together, with what they were accustomed to in ordinary concrete foundations and concrete piers, and that really did not give the concrete in its ordinary state a fair test. Lately he had seen several very large reinforced-concrete works in course of construction, and the extreme care that had been taken was quite remarkable. That was necessary when it was remembered that many of the bars forming the reinforcement came within 2 inches of the surface, and the great care that had to be adopted both in putting the material together and in selecting the material itself could be quite understood. But he thought that reinforcement was used in many cases in which there was no practical advantage in it. The object of reinforcing concrete was presumably to obtain a structure that was cheaper than one in which a larger mass was used to obtain the same strength. If, in order to obtain a certain strength, a form of construction was adopted which cost a considerably larger sum of money than would otherwise be necessary, then, from a practical and commercial point of view, he did not see that the great advantages claimed were obtained. He admitted that

Mr. Elliott-Cooper. in many of the cases he had seen, where space had been of great importance, the results obtained could not have been improved upon by the use of other materials, so far as he was able to judge. Taking the illustrations in the Paper, Fig. 2, Plate 5, showed an arched bridge of a span, from centre to centre, of 100 feet, although the clear span really was about 85 feet, the angle of skew being 45°. There was nothing very remarkable about that. The thickness of the arch at the centre was given as 3 feet 9 inches. If the same bridge were built of brickwork the dimensions referred to would be just about what would be adopted. Rather curiously, he had been called in to advise upon a bridge abroad, where they had built the arch slightly less than 3 feet thick at the centre for a bridge of very nearly the same span as the one illustrated, and he recommended that as they were going to build six other bridges of the same kind, they should increase the thickness of the arch at the centre to 3 feet 9 inches. He also mentioned in his report that according to the ordinary English practice it would be 3 feet 6 inches, but that, as the bridge was in a foreign country, and he did not know the degree of reliability of the workmanship, he had given a small margin. It therefore appeared that a similar bridge to that shown by the Authors, constructed of stone without any reinforcement, would be amply strong with the same dimensions, and accordingly he could not see the use of putting in a series of bars at the top and bottom of the arch to make it stronger. As illustrating that point, the other day he was looking at a building in which there was a floor, with a slightly groined roof, of 40 feet span in one direction and 32 feet span in the other, and the thickness of the floor, which was calculated to take 500 lbs. per square foot, was 8 inches. He thought that was somewhat thin, but if it would stand, it was a great achievement for reinforced concrete. Where a structure was of the full thickness, however, and could be built of ordinary materials, he could not see the advantage of reinforcing it. The box abutment specially referred to in the Paper was the ordinary form English engineers were accustomed to, but instead of a flat slab—which he noticed was 3 feet thick for a span of 13 feet—an ordinary arch would have been put in. He did not believe there was very much saving in the actual quantity of material, and, if so, there could be no possible advantage in reinforcing that thickness of concrete with steel bars. Coming to what he considered the most curious of the designs shown, the ordinary single-line bridges, they appeared to him to be ordinary steel structures encased in concrete, and he could not think that the mere encasing of the rolled joists or built-up girders in concrete would add materially to the

Mr. Elliott-Cooper.

strength. It might add stiffness, but he did not think it could add to the permanent and actual strength. In one of the cases where a special form of structure was employed (Fig. 5, Plate 5) it was stated that that form was adopted for the purpose of getting a very small space between the rail-level and the underside of the girder. The depth was given as 2 feet 6 inches. He had built a bridge over the Midland Railway where he was limited very much in depth, and he managed to get across with 1 foot 3 inches. Therefore he could not quite see where the advantages of reinforced concrete of the kind shown in the Paper came in, or why, if headway was a special feature, it could not be managed with less depth than 2 feet 6 inches. It was stated in the Paper that in cases where it was very important that the structure should be watertight a particular blending of material had been used, but it seemed to him that, if the reinforcement principle was used at all, it was a question not whether water was coming down on to the heads of people below, but whether water was penetrating into the material; because if it did penetrate the steel bars would corrode sooner or later, and if the bars corroded and continued to corrode, the structure, so far as its strength depended on the reinforcement, would fail. He considered that all reinforced concrete ought to be perfectly watertight if the material of which it was made was to be permanent. With regard to the culverts, he had taken out the cost of culverts of the ordinary circular form and found they came out almost exactly the same; so that even in that case he was sorry to say he could not quite see where the economy of reinforced concrete of that description occurred.

Mr. Hudleston.

Mr. F. Hudleston noticed that the Authors stated that the structures described were as cheap as ordinary masonry, but, as Mr. Elliott-Cooper had shown, there was not very much difference between their cost and that of brickwork. He had compared the Sangamon River bridge with what he thought would be the modern price of the magnificent Wharncliffe viaduct on the Great Western Railway at Hanwell, built about 70 years ago. The two viaducts were very similar in height, the arches of the English one being about 65 feet in span, while the American viaduct was about 85 feet. Brunel's viaduct was built entirely of stock bricks, and was still a very fine example of an English viaduct; it looked as sound as ever from the outside, except that the westernmost arch seemed to have had a slight settlement at one period: possibly the abutment had yielded a little bit. He made out, from external dimensions, that that viaduct at modern prices, unless the foundations were something extraordinary, would cost about £100 per lineal

Mr. Hudleston. yard, while the American bridge cost £125—assuming that each abutment was about 100 feet long. Of course, in America prices were higher than in England, but he thought it would be generally admitted that if things were compared properly there was not much to choose between those two viaducts in price per lineal yard. Then with regard to the durability, it was said that reinforced concrete was permanent. No doubt it was extremely lasting, but he did not think anybody would care to say that a reinforced-concrete structure could have the same life as a brick structure such as Brunel's viaduct. There was a great deal to be learnt about reinforced concrete. It was well known that concrete had a tendency, especially when rich, to develop hair cracks, and it might be that something of that sort would happen at some future time in reinforced bridges. For plain arched viaducts he thought nothing was gained by using reinforced concrete instead of ordinary bricks, which it was known were very durable and cost next to nothing for maintenance. Again, the Authors said that, as compared with steel, reinforced concrete was altogether superior, as it required no maintenance. Of course it did not require painting, and probably it had a longer life than steel girders; but after all, the comparative cost of a brick-arch bridge and a steel-girder bridge of the same span showed that the brick arch would come to about £100 or £150 more than the other; the one would be between £400 and £500, and the other between £500 and £600. When considering maintenance, it was necessary to take into consideration the extra interest on the capital. That extra £100 meant £3 or £4 a year, and £3 or £4 a year would go a long way towards a sinking-fund for a steel bridge. Even in brickwork it was necessary to allow something for repointing occasionally. He did not think, therefore, that the comparison, which at first sight seemed a fair one, was quite as reasonable as it ought to be. If a thing had to be built cheaply, then the steel girder certainly held its own; but taking the real cost—the interest on the capital expenditure plus the cost of maintenance—he did not think there was much difference to be found between a reinforced-concrete bridge of ordinary span and a steel one. With regard to the other examples in the Paper, on looking carefully it would be found that the structures consisted of steel girders buried in concrete. That rather gave the case away for reinforced concrete for underbridges and girders. In England the Board of Trade would not allow it to be used, although they might do so in time. It would be seen that on the Wabash Railroad they had not really had the courage of their convictions, all their large bridges being of steel girders buried in concrete. With regard to the floors, he

Mr. Hudleston. thought they were very ingenious and ought to have a much longer life than the steel floors used in England. Mr. Elliott-Cooper's point about water percolating through was a very good one. It seemed to Mr. Hudleston that if a certain quantity of bituminous substances was mixed with concrete, as apparently had been done here, it must interfere somewhat with the strength of the concrete. A great deal had been written and said about the design of reinforced-concrete work, but he thought engineers were still very much in the dark as to what really ought to be done. There were very few experiments to serve as a guide in forming a theory for calculating the reinforcement. In steel-girder work there was a vast amount of experience with regard to the theory of stresses and so on. A great deal more had to be learnt with regard to reinforced concrete before it could be said definitely that any set of rules could be really applicable to all sets of circumstances. Looking at the figures given by the Authors at the beginning of the Paper, there were one or two things a little inconsistent. The unit stress allowed was fixed by a rule well known in America. On p. 134 it was said that the maximum allowance for steel, including impact, live load, and dead load, was 18,000 lbs. or about 8 tons per square inch. That gave an old limit for ordinary mild steel—about 8 tons where all the load was dead and 4 tons where all the load was live. A little later on the Authors said that a 47-ton high-carbon steel, with a breaking-stress of about 105,000 lbs. per square inch and an elastic limit of 66,000 lbs. per square inch, was used. He did not see the advantage of having such a high-carbon steel, and thought that an ordinary mild steel, such as was used in England for structural purposes, would serve all the Authors' purposes. The 18,000 lbs. was practically the kind of load used in America for mild steel, and he would like to have an explanation as to why the Authors used a high-carbon steel when their stresses were 8 tons for dead load and 4 tons for live load. The limit of 18,000 lbs. mentioned in the Paper corresponded practically with the Board-of-Trade regulation. Taking into consideration the dead load and everything else, in the main girders of an ordinary English railway-bridge it was hardly possible to get much more than 6½ tons, even with 18,000 lbs. as the limit.

Mr. Meik. Mr. C. S. Meik considered that the information given in the Paper was hardly sufficient to enable any reasonable criticism to be offered with regard to the various designs that were shown, but on carefully looking through the Paper one or two things had occurred to him much the same as had occurred to Mr. Elliott-Cooper. The question of the arch of the Sangamon bridge was one in particular.

Mr. Meik. Reinforcement was quite unnecessary for that bridge if it was made of good concrete, because the arch was amply strong enough. With regard to the Forest Park bridge, No. 11, the floor was shown as being of reinforced concrete 8 inches thick, carried upon rolled beams 18 inches apart between centres. At the first blush it would appear that reinforcing was quite unnecessary there, but so little information was given that it was impossible to check the calculations, and possibly there was some reason for it which the Authors had not stated. Coming, however, to the question of stresses, there was a little more to go upon. Taking, for instance, the steel, which the Authors said was to be stressed up to 18,000 lbs. per square inch, with an elastic limit of 55,000 lbs. per square inch, that was a higher class of steel than was customarily used for reinforced concrete in England, or even in France; but it had appeared to him for some time past that the use of such steel was a step in the right direction. For reinforced concrete a high-class steel was required, because it gave a higher elastic limit. In most reinforced structures—beams or floors—the limit of elasticity of the steel was really the limit of the safety of the structure, because once the steel began to stretch and exceed the elastic limit the beam invariably failed, even although the concrete was not stressed up to its ultimate strength. The stresses used in England ranged from 15,000 lbs. to 17,000 lbs., with an elastic limit of 34,000 lbs. per square inch, giving a factor of practically 2, taking the elastic limit of the steel as the point of failure of the beam, which was very near the mark. With regard to the concrete, the Authors gave the working limit on the outside fibres of the beam as 800 lbs. per square inch with a mixture of 1:2:4, but nothing was said about the ultimate strength of the concrete. It might be, as he had found, that a mixture similar to that gave as little as 0·53 ton per square inch, or the same mixture might give 1·64 ton per square inch, a variation in the ultimate strength of between 300 and 400 per cent. He thought that pointed to the fact that in all specifications for reinforced concrete there should be some statement as to what the strength of the concrete should be, because it must be apparent to anyone that if concrete could be made with such extreme variations the factor of safety was practically of no use. In connection with some reinforced-concrete work his firm had carried out at Swansea he had made tests of the compressive strength, and he had found out what had doubtless been known to a great many people, especially in America, that the heavier the concrete was per cubic foot, the higher was its compressive strength. He might say that, with regard to the figure he had quoted of 0·53 ton per square inch, as the ultimate compressive strength, that concrete

Mr. Meik.

weighed 126 lbs. per cubic foot. The concrete giving heavier stress of 1·64 ton weighed 149 lbs. per cubic foot. All the other experiments carried out showed that the strength increased with the weight, but of course in a far greater ratio. Since he made those tests he had had some more cubes made and tested; he had succeeded in obtaining a concrete weighing 158 lbs. per cubic foot, and he believed that in America they had succeeded in getting concrete which weighed 160 lbs. per cubic foot. The heavier the concrete, the denser it was; and the denser it was, the stronger it was bound to be. The blocks referred to were made of granite and sand, and if it were possible by improved mixing to make them as heavy as the granite of which they were composed they would presumably have the same strength. That would probably never be attained; nevertheless an endeavour should be made to obtain as heavy a concrete as possible. Heavy and dense concrete had the advantage of greater impermeability, which was of great importance in a material such as reinforced concrete. It would obviate a great deal of the risk, advanced by Mr. Elliott-Cooper and the last speaker, of moisture getting at the steel. The Authors stated that they preferred a wet concrete to a dry concrete and proceeded to give the results of some tests by Mr. G. W. Rafter. But unfortunately the tests did not bear out the statements of the Authors, because the record on p. 136 showed that concrete made of a dry mixture withstood 2,470 lbs. per square inch, whereas concrete made of a wet mixture withstood only 2,180 lbs. per square inch, showing that the dry mixture was the better of the two. His own experience was that a dry mixture was better. There had, of course, to be a certain amount of moisture, but provided that was sufficient to work, it was better to use a dry mixture than a very sloppy one. The more water there was put in, the less chance there was of getting a really dense mass afterwards. The Authors also made some remarks about plain bars as against corrugated and ribbed bars, and said that some form of ribbed bar was to be preferred to plain bars. He went into that question some little time ago and had some correspondence with Mr. Barclay Parsons, M. Inst. C.E., of New York, who told him that he was experimenting with bars and strongly advised the use of the ordinary plain smooth bar. Since then Mr. Parsons had given the results of his experiments in a Paper read before The Institution on the New York Rapid-Transit Subway. Mr. Parsons there pointed out that slightly rusted rods were better than clean rods, a conclusion at which Mr. Meik himself had arrived. The plain rods rusted stood 437 lbs. per square inch before being drawn out, twisted rods stood 509 lbs. and enlarged rods 637 lbs., showing

Mr. Meik. a slight advantage to the twisted rods and the enlarged rods; but Mr. Parsons made the following observation:[1]

> "In all cases [of the twisted and enlarged rods] the concrete block was split, indicating a wedge-like action of the enlargements and the twists. If a mechanical bond is desired in addition to adhesion it can be better obtained by a square shoulder, such as a nut on the end of the rod, than by any shape that tends to split the enveloping mass."

With that conclusion he cordially agreed. He had never found any difficulty, provided there was a sufficient length of rod embedded in the concrete to get an adequate bond. Failing a sufficient length, it was necessary to rely on a mechanical device like a square shoulder or a nut.

Mr. Thorpe. Mr. W. H. Thorpe observed that there seemed to be a considerable divergence of opinion about the use of a deformed bar or a plain round bar, but he thought it was largely a question of design whether the round bar would serve its purpose sufficiently well. If due care were not exercised to ensure that the grip required of the concrete was of moderate amount and that the maximum grip at the end of a bar in particular cases was not excessive, there might be some failures resulting. Taking the case of a simple beam of uniform depth, loaded in the middle, with bars parallel to the bottom edge, the rate of grip required of the concrete along the bar to hold it, would be uniform. If the load was distributed, the maximum grip required at the end of the bar was twice as great as the mean grip. In the case of a vertical wall sustaining water-pressure the maximum grip on the rod would be three times as great as the mean. This was quite obvious, and he was simply calling attention to it. If it should happen in any piece of work that less than sufficient care was given to the maximum stress which might be developed on a short length of the rod, some inconvenience might result. A deformed bar had the advantage that it would hold perhaps 50 per cent. better than the plain bar; thus there was a wider margin for carelessness. With proper design, and avoiding the use of large rods—which necessarily required a considerable length of enshrouding concrete to grip them—there should be no difficulty, as a rule, in ensuring a proper hold upon the bars. With regard to the question of wet versus dry concrete, the Authors stated that the advantage of dry concrete over wet concrete in point of strength was small; on the figures they gave it was about 14 per cent. He had looked up the experiments referred to, contained in the admirable volumes issued by the United States Government on Tests of Metals and Other Materials, copies of which were in the Library, and in the 1898 issue he found about 250 pages dealing with Mr. Rafter's and other tests. Looking through those tests in some detail, he concluded that, although the summarized result was to show that dry concrete had but a moderate advantage over the wet mixture, yet in particular series of experiments the advantage of the dry concrete over the wet might be as much as 50 per cent. He thought the conclusion of the Authors with regard to dry versus wet mixtures was not that which he would care to accept. With respect to the question of durability, he would refer to two structures of considerable age in England, the work of Mr. W. B. Wilkinson, of Newcastle-on-Tyne, whom he might be allowed to claim to be the pioneer, not only in this but in any other country, in the matter of reinforced concrete.[1] Mr. Wilkinson took out a patent, No. 2,293, in the year 1854, and in that patent showed a method of construction illustrated by the two drawings Mr. Thorpe had placed upon the table, tracings made about that time and in accordance with which the specification-drawings were prepared. They showed a method of construction which was reinforced concrete pure and simple. That Mr. Wilkinson had a knowledge of the principle underlying the work was shown by the statement in his specification referring to the reinforcement used : "The under edge of the same is set on the crown of centering, or nearly so, so that in this low position it may act with the more power as a tension rod to the floor." He clearly knew what he was about, and an examination of the drawings would show that his reinforcement was placed in a manner that it would be difficult to improve upon even now. He used wire-ropes, and Mr. Thorpe had been told that the reason was that at that time there was difficulty in getting round rods of sufficient length. Of the examples of Mr. Wilkinson's work referred to, still in existence, one dated back about 22 years, and consisted, on the authority of Mr. Hubert Todd, who made the working-drawings, of a floor about 14 feet in span, 12 inches thick, and loaded with 13 cwt. per square foot (by a baker's oven), to be found in Northumberland Court, Blackett Street, Newcastle-on-Tyne. The other case was a cottage in Wilkinson's storeyard, Ellison Place, in the same town. It was difficult to give the precise date, but it was probably built about

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[1] Minutes of Proceedings Inst. C.E., vol. clxxiii, p. 122.

Mr. Thorpe.

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[1] Further inquiry shows that a near approach to reinforced concrete was developed about 1850-5 by a Mr. Lambot-Miraval, who constructed a boat 3·30 metres long by plastering Roquefort cement upon a skeleton network of iron and wire. See A. Delesse, "Matériaux de Construction de l'Exposition Universelle de 1855," Paris, 1856 : to be seen in The Institution Library.—W. H. T.

Mr. Thorpe. 40 or 50 years ago. He was assured by Mr. Todd, and by Mr. Steele, the City Engineer of Newcastle-on-Tyne, that that building was in fairly good condition. Mr. Todd said that it showed absolutely no cracks, and Mr. Steele stated that there was little or no evidence of disintegration such as would be expected if rust were acting on the reinforcement under the concrete.

Mr. Steinberg. Mr. H. E. STEINBERG remarked that there was no engineering subject that needed discussion so much as reinforced concrete. At present in England attention to the matter seemed to be confined to two classes of people, those who had to practise it and those who wrote and talked about it; and although reinforced-concrete work to the value of probably a million sterling, was done in this country every year, the English text-books and discussions on the subject led one to believe that the question was still in its infancy. With regard to the Paper, he wished first to refer to the use of deformed bars. The Authors said that many American engineers did not look with favour upon the practice of using plain bars as reinforcement in concrete. On the other hand, many American engineers, to judge by American literature, did use plain bars in reinforced concrete, and practically all French, Austrian and German engineers did the same, and he hoped that all English engineers would do so. Deformed bars seemed to him quite unnecessary. Deformed bars such as those referred to in the Paper depended as much on adhesion for the transmission of their stress as plain bars did. If the adhesion were taken away, the little shoulders on the bars—which were said to be practically at right angles to the axis of the Johnson bar, but which in the examples on the table were practically at 45°—would have to transmit to the concrete a direct compressive stress of about 1,600 lbs. per square inch, a figure far in excess of anything which should be allowed. He wished to endorse what had been said already with regard to the reinforced-concrete arch shown in Figs. 2, Plate 5. It was not a reinforced-concrete design at all, and such arches had no need of reinforcement. The resultant of the dead load and the live load in any possible position would never pass outside the middle third of the section. The armouring could therefore never be in tension, and was quite unnecessary. A great deal might be said also against the design of most of the other work. For instance, in the culvert in Fig. 11, if the bars in the gusset were in compression, as presumably they were, then the concrete at the other end of a section taken diagonally through the corner, must be in tension, and no armouring to resist this effect was shown. The whole dimensions

Mr. Steinberg.

of the structures were too large for reinforced concrete. The Authors spoke of slabs 3 feet thick. In a slab of that thickness four-fifths of the armouring was carrying the slab and one-fifth was carrying the live load. All the figures as to cost were, therefore, as a basis for comparison, in his opinion, quite valueless. Reinforced concrete would certainly never have justified itself or become so popular if it were designed on lines such as those shown by the Authors. With regard to formulas, it was said that the parabolic formula of Professor Talbot was used. Parabolic formulas were very cumbersome to work with, and the extra accuracy, in the second place of decimals, might be considered unnecessary. It was almost impossible in reinforced-concrete work to design precisely; the utmost that could be done was to design by a kind of guesswork and then check the design to see whether it came out correct. There was a graphical method of working out directly the quantity of steel required, but its use was not widely known. The percentage method, mentioned in the Paper, was also very cumbersome to work with. With regard to the steel girders reinforced by bars placed alongside them, it would be quite impossible to calculate them by any formula. In the second Appendix, on p. 146 a Summary of Building Regulations governing the use of reinforced concrete in the United States was given: the point he wished to touch upon related to the bending-moment of continuous beams. In New York, Buffalo, and San Francisco it was $w\,l/8$ and in Cleveland $w\,l/10$. It was quite permissible to legislate about the stresses to be allowed. One State might fix 16,000 lbs. and another State 15,000 lbs. per square inch on the steel; but no State could legislate directly about bending-moments. Once the span and the loading were fixed the bending-moment was fixed. In a simple case of an ordinary beam resting on two supports it would be absurd for New York, Buffalo, and San Francisco to say that the bending-moment should be $w\,l/7$, and Cleveland to say $w\,l/9$; it would still be $w\,l/8$, whatever they might decide. When the span was continuous it was still quite impossible to govern bending-moments by legislation. The monolithic nature of reinforced concrete rendered every slab and girder passing over supports a continuous beam, and although continuous beams were very troublesome to calculate, it was necessary, if the armouring was to be properly arranged, to recognize that property of continuity and to calculate the armouring accordingly. Referring to the regulations of different countries, in Germany it was stipulated that continuity should be recognized, and only if the beam could not be calculated exactly according to theory as perfectly

Mr. Steinberg. continuous should $wl/10$ be taken in the span and $wl/8$ in the supports. In the Report of the Royal Institute of British Architects it was stated that the bending-moment in the span was $wl/24$, and over the supports $wl/12$. At Buffalo they gave $wl/8$, which was probably in the span, and Cleveland gave $wl/10$. It seemed to him the whole thing was quite unnecessary. If the methods suggested in the German regulations were adopted, and the moments were calculated as accurately as possible, according to the accepted laws of mechanics as applied to continuous girders, or if, when the problem was too complex for this to be done, then high fixed values for the bending-moment were taken, it would meet every case.

Sir H. Tanner. Sir **Henry Tanner** remarked that his experience of reinforced concrete was limited to buildings, and therefore his remarks would hardly bear upon the matter from the railway-structure point of view. His attention was first directed to reinforced concrete about 6 or 7 years ago, and arose mainly from considerations of economy. He went to see a building put up by Messrs. Hennebique, through Mr. Mouchel, at Newcastle, and it struck him as being a very economical building, while taking up very little space, the walls being so thin that very little floor-space was lost inside, which was, of course, a great advantage. He went into the matter and persuaded his Commissioners to allow him to carry out the Post Office in reinforced concrete, and the result that a saving of about £60,000 was effected on that building. The carcase cost about 2*d.* per cubic foot, whereas, if done in steel it would have cost 4*d.* The other things, of course, were practically the same. There was some stone in the street fronts, but the greater part of the structure was concrete. The walls were 7 inches to 8 inches thick. If the building had been of brickwork he believed the thinnest walls would have been 18 inches and at the bottom the walls would have been about 3 feet 6 inches at least in thickness. The piers were no larger than they would have been if they had been of steel, and probably smaller, because it would have been necessary to encase the steel in concrete. So far his experience had been extremely satisfactory, and he had gone on to other buildings. He had found the concrete to be quite waterproof. In the basement there were walls 25 feet high and 7 inches thick against the earth, but no water came through, and the place was perfectly dry. The floor was only 5 inches thick and the water-level was about 18 inches below it. The only place where water came into the basement was where the levelling-pegs were left in; wherever a wet spot appeared in the floor it

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Sir H. Tanner.

was known that a levelling-peg was there. As to the mixture, he thought that rather depended on the amount of reinforcement. Where the reinforcement was heavy, the mixture had to be thinner, but in course of time that corrected itself. So far as he had noticed, the dry mixture became harder more quickly and bore a load at an earlier date, but the wet mixture quite made up for it and probably surpassed the dry mixture in strength eventually. With regard to rods, he always preferred round ones, which he thought could not be beaten. He did not care for corrugated rods of any sort. The extreme tension which required deformed rods to prevent slipping should not be reached. He did not think there was any fear, if the concrete was properly made, of any rusting occurring. The rods he had used had been taken generally out of the yard, where they had stood in all weathers for a considerable time, and all that had been done was to brush off the rust and anything that was loose. He had had holes cut in the concrete and had always found the steel to be as blue as if freshly manufactured. He thought the great point about reinforced concrete was economy, which governed everything.

Mr. Dunn.

Mr. W. Dunn sympathized with a great deal of what had been said with regard to the examples shown not being very economical structures. They seemed to him to have been designed by an engineer who desired to try a new method of construction, but who was not prepared to go so far as a specialist in reinforced concrete would go. In Paris a little time ago he had occasion to examine a bridge over the canal St. Martin, which had a span of 27 metres. The arch had three semi-articulations, and the thickness was 0·45 metre. That was in marked contrast to the arch shown in the Paper. As had been rightly said, the concrete in these arches would be amply sufficient to do all the work, and the steel could only have been put in to resist cracking due to temperature-stresses or other causes. It was said that a very high stress was allowed—up to 800 lbs. per square inch—in the concrete. Looking at the sections of the arches he quite failed to see how such a stress could possibly be developed. He thought it was coming to be more and more the custom to limit the stress in the steel to 16,000 lbs. per square inch. There was a good reason for doing that, because if the stress in the steel was very high the concrete tended to open. If the stress in the steel was 18,000 lbs., the stress in the concrete at the same level was approximately about 1,200 lbs. per square inch, and any concrete would crack at that. The stress in the concrete 2 or 3 inches below the steel was even higher. Therefore there was a strong reason for keeping the stresses in the steel at a low limit. He agreed with previous speakers that it seemed peculiar to use high-carbon steel with an ultimate stress of

Mr. Dunn. 105,000 lbs., and yet keep the stresses as low as 18,000 lbs. Naturally engineers feared to adopt a material about which there was some question as to its permanence. In many cases it was a much more permanent material than steel could possibly be. In Paris he had seen, in bridges over a railway, steel girders side by side with reinforced-concrete girders, which had been in for 12 to 14 years. The steel girders had been frequently painted, and yet had rusted very badly. The reinforced concrete, he was assured by the engineer in charge, had received no attention, and, as far as could be judged, it required none, being in perfect condition. With regard to the artistic effect of a bridge, a large mass of plain concrete in a wall had naturally a rather unpleasant colour, but when it was put into a bridge, whether an arch or a plain girder-bridge, there was a chance of making it much more artistic and agreeable than a lattice girder or other form of steel girder. He was only sorry the members had not an opportunity of seeing illustrations of a number of charming bridges that had been built in France and Germany, mostly arched bridges, which structures to his mind offered a splendid field for the use of reinforced concrete. One speaker had mentioned the Report of the Joint Committee of the Royal Institute of British Architects, and stated that the bending-moment should be $wl/12$ in the span and $wl/24$ in the support. He thought the speaker was wrong in that. The Committee suggested that $wl/12$ would be a fair allowance to take in a continuous beam in the centre of the span, and naturally the allowance differed with the distribution of the live and dead load. What the Report did say was that when continuity was calculated, every proper provision must be made for the moment at the support, which seemed a very natural thing to say. It did not lay down what the bending-moment over that support should be.

Dr. Brightmore. Dr. A. W. Brightmore remarked that it had already been pointed out by more than one speaker that advantage had not been taken of the reinforcement in the designs to reduce the dimensions. It was not quite clear in Fig. 3, Plate 5, why a reinforced abutment should be adopted for a plate-girder bridge. In Fig. 8 plain concrete abutments were adopted for a reinforced girder; also the two piers were shown as being reinforced. That presumably was a reinforcement for compression, because the girders were not shown to be continuous. It was questionable whether any advantage was gained by that reinforcement. It seemed a logical thing to introduce steel bars where there was tension in concrete, but it was not so clear why steel bars should be introduced to assist in the compression. A good many experiments had been made which showed that there was more uncertainty about benefit being derived from reinforcement in compression than in tension. If the unit stresses referred to in the Paper were actually adopted, the factors of safety would be reduced considerably below what were considered desirable in ordinary engineering practice. It was mentioned that the working-stress in the concrete in compression in cross bending was taken as 800 lbs. per square inch—rather over 50 tons per square foot, and he thought that was a stress which in an ordinary engineering structure would not be looked upon with complacency. It was also stated that for direct compression it was reduced to 600 lbs. per square inch; but even that worked out at about 38 tons per square foot. No doubt a good deal of the saving that was obtained by using reinforced concrete with such unit stresses was at the expense of reducing the factors of safety beyond what was considered desirable in ordinary structures. Dr. Brightmore.

Mr. Marsh. Mr. C. F. MARSH thought it was a pity that the Paper should have been selected to show what could be done in reinforced concrete, because there was hardly a fair example of that material in the whole communication, and he thought it would have been much better if a Paper showing what could really be effected by the use of reinforced concrete had been brought forward for discussion. With regard to the remarks that had been made comparing reinforced concrete with brick and concrete, he hardly thought them fair criticism. Reinforced concrete was reinforced with iron or steel, and it was not the mixing of the concrete which gave strength; that only gave strength in compression. Mr. Elliott-Cooper thought that if any water soaked through the concrete it would attack the steel and cause rust. Even if a certain quantity of water did go through—and Mr. Marsh did not think it would, in properly-constructed reinforced concrete—unless carbonic acid gas and oxygen also gained access to the steel, no rusting could occur. The stresses of 800 lbs. and 600 lbs. per square inch, given in the Paper, were altogether beyond what ought to be allowed, and were very high for American practice. He thought the use of the compressive stress of 800 lbs. per square inch on the concrete must have arisen from the fact that Talbot's formula had been used and 18,000 lbs. per square inch assumed for the stress on the steel; and that on working out the formula for 1 per cent. of reinforcement it had been found that a compression of 800 lbs. per square inch on the concrete was required, and therefore this stress had been assumed. The shearing-stresses were extraordinary. It was assumed that if the concrete was reinforced with steel against shearing—and presumably the Authors took the steel into account in their calculations for the shearing resistance—more stress could be allowed on the concrete in shear. Taking what the steel would give as an addition to the

Mr. Marsh. shearing resistance of the section, he did not see how any more resistance could be allowed for the concrete. With regard to high-carbon steel, he was entirely at variance with the Authors. Mr. Dunn had said very much what he himself would say on this question. It was extremely dangerous to use steel under high stresses on the tension side of the concrete. With regard to corrugated and other deformed bars, there was no doubt that they did give additional bond resistance; but the question really was whether plain bars were sufficient for the purpose, and he thought it had been clearly shown time after time that plain bars, secured in a proper manner at their ends, gave sufficient resistance against slipping. As to the mixing of concrete, he thought it best to mix it neither too wet nor too dry, and to have a medium mixture. With reference to the arched bridge, he thought that it would have been much better if some of the reinforcement had been taken out of the arch and put into the parapets to reinforce them against temperature-stresses, which had a marked effect on long lengths of concrete.

The President. The PRESIDENT, in calling upon Mr. Matthews to reply, remarked that he thought the discussion had been very interesting, notwithstanding the last speaker's deprecation of the reading before The Institution of a Paper which was the subject of vigorous criticism. He quite understood that there were finer examples of reinforced concrete, and he would suggest that Mr. Marsh might offer a Paper upon more advanced specimens of reinforced-concrete work. The Institution was always glad to have descriptions of works, particularly in other countries, and to have the experience of people who were groping their way, because all engineers had to do that in some degree. Therefore he congratulated the meeting on a very interesting discussion on a subject which was exercising the minds of a large majority of the members and was interesting to all, and a subject upon which the Council were spending a great deal of time and money in investigation and in carrying it farther than perhaps anyone in the room that evening knew.

Mr. Matthews. Mr. MATTHEWS, in reply, stated that his co-author and he were not in agreement with regard to the whole of the structures mentioned in the Paper. He certainly thought that in some of the structures there was far too much material. The thickness of concrete in the arch of the Sangamon River bridge, for instance, was probably greater than it need have been. But the object of bringing the Paper forward had been to show that in America, in railway engineering, reinforced concrete was being used because

Mr. Matthews.

it was considered that it resulted in a marked saving. On the Wabash Railroad it had been used extensively. With regard to Mr. Meik's remarks, he was quite sure that the necessity for a high class of steel for reinforcing bars was very apparent, and he did not agree with some of the other speakers' remarks on that subject. There had been a good deal of discussion on dry versus wet concrete. He preferred wet concrete, for the reasons advanced in the Paper. He had carried out a good deal of reinforced-concrete work—walls, flat roofs and beams—and his opinion was that a better and denser concrete was obtained by making it wet, and, what was more important, the reinforcement was better embedded than with a dry or even a plastic mixture. With regard to the particular bars shown on the table, he was not committed to any particular system of reinforcing bars, but, as stated in the Paper, in America engineers looked with some disfavour on plain bars, preferring some form of deformed bar, such as a bar with a ribbed device, or a twisted bar, or the Thacher bar. With respect to the question of economy, he had himself effected a saving of 30 per cent. by using reinforced concrete in the construction of retaining-walls at Bridlington instead of brick or monolithic concrete, and was aware of experiences of other engineers which corroborated such economy. As to the durability of reinforced concrete, which had been referred to by several speakers, none of them had questioned the durability of properly-gauged concrete; and with regard to the possible corrosion of the steel in the concrete, experience had shown that, if a dense mixture was formed, and the aggregate was suitable, there was no fear of corrosion taking place. He had occasion, some little time ago, to remove a portion of a concrete-in-situ sea-wall at Bridlington which had been constructed 25 years previously, and in doing so he laid bare several chains which had been inserted in the wall when it was built. He found that the chains were quite bright to within ½ inch of the face of the wall. He also removed recently a concrete roof constructed 18 years ago, and partially reinforced by 6-inch by 4-inch tee-bars. He had found the tee-bars perfectly bright. In further support of his view he would refer to the results of tests carried out by him in 1908[1] in order to ascertain whether corrosion of steel reinforcement in concrete took place. The advantage of using reinforced concrete in the construction of bridge-floors was

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[1] "The Corrosion of Steel Reinforcement in Concrete." Transactions of the Society of Engineers, 1909, p. 61.

Mr. Matthews. that corrosion of the floor-plates was avoided. He did not agree with Mr. Hudleston that very few experiments had yet been carried out with reinforced concrete.

⁂ Mr. Cunningham's reply will be found at the end of the Correspondence.

Correspondence.

Mr. Colman. Mr. S. M. **Colman**, having watched for some years the struggle for recognition which reinforced concrete was making, wished to notice some conditions of its use. First, the methods adopted by specialists could not be protected. If a monopoly or close ring of practitioners were formed, or if legislation requiring severe testing and substantial guarantees were introduced, the unqualified engineer would soon abandon its use. Its intrinsic merits would then raise it to the level which it undoubtedly deserved. Then, it was a material which lent itself too readily to imitation; the apparent simplicity of some of the great works accomplished in reinforced concrete had led many men of doubtful ability to emulate experts by producing ridiculous or over-bold designs which had been a factor in the high percentage of failures. Again, its efficiency depended to a very large extent on perfect workmanship; this perfection often being required in details known only to the designer, the least relaxation in the duties of supervision perhaps meaning failure. This dependence on workmanship, rather than on any factor of safety, was a good reason why the designer should himself take charge of the construction and not depute this all-important task to a subordinate. The failure of a structure, due to its faulty design by a self-constituted expert, tended to destroy confidence in good work by specialists. The direction which discussions on reinforced concrete took showed such want of confidence in each other among its most sincere advocates, that an impartial engineer was justified in condemning them all for that reason alone. Then again it had its legitimate and its illegitimate uses, and examples of the latter were the more frequent. This was undoubtedly due to the fact that those who had arrived at or assumed the position of experts were inclined to recommend its use in all places where it could be, rather than in those places only where it should be, employed. The former being much more numerous than the latter, many costly

Mr. Colman.

experimental structures were built; and these unhappy attempts to popularize the material reacted on the honest work of specialists engaged in its legitimate applications.

Mr. Gilbert.

Mr. **William Gilbert** thought it would be useful to have a further description, or a drawing, of the corrugated bars used for reinforcement. It appeared that small rods, such as ¾-inch square bars, were sometimes placed as much as 2 feet apart. The maximum permissible spacing of the bars should depend on the thickness of the concrete slab in which they were used, and it would be interesting to learn the Authors' practice in this matter. On p. 139 a reference was made to bridges crossing streams, whose floors, made of 1 : 2 : 4 concrete, were not necessarily made watertight. It appeared to him that continued percolation of water through the floor would in time attack the steel embedded and cause a breakdown. With reference to the formula of Professor A. N. Talbot, given in Appendix I, this was stated to be based on the assumption that the curve connecting stress and strain, for a bar of concrete under compression, was a parabola. For ordinary working-stresses, the portion of the parabola in use nearly coincided with the straight-line relation between stress and strain assumed by the Royal Institute of British Architects Committee, so that the two formulas gave approximately similar results, but the latter was more easily derived. For results above the ordinary working-stresses, and up to the elastic limit of the steel, Professor Talbot's formula would apparently give the more accurate results. He had had to deal, on several occasions, with the design of cement-stores, where the pressure of the cement against the walls was of considerable amount. Using brickwork, and making the usual assumption that no tension was carried by the mortar of the joints, involved relatively very thick walls, and an examination of existing cement-stores, by the application of this assumption, would show that the factor of safety was often very small, perhaps 1¼ to 1½. The use of reinforced concrete enabled much cheaper walls to be built, and the factor of safety could be permanently 4. The Authors, in his opinion, were to be congratulated on the results given in the Paper, and on the courage and enterprise shown in pushing forward the use of reinforced concrete for railway work.

Mr. Goodrich.

Mr. **E. P. Goodrich**, of New York, remarked that reinforced concrete was used so widely on railways that it had come to displace in large part steel and timber construction for locomotive-houses, coal-pockets, stations, and other structures, as well as for bridges, culverts, and pipes, such as those described in the Paper. The data in regard to the last items were of interest largely because

Mr. Goodrich. of the considerable period during which the Wabash Railroad had made use of this material, as well as because of the unique features embodied in the designs described. Reinforcing bars which afforded a mechanical bond were being used in less proportion as time went on. The actual weight sold of such bars was increasing, but, according to the best information obtainable by him, not as rapidly as the total weight of all bars sold for reinforced-concrete work. The cost data given were interesting, but such information must be interpreted in the light of local conditions, which invariably modified considerably the forces of both material and labour. Another interesting point was the method of waterproofing the floor of the bridge over Junction Avenue, Detroit, by the incorporation of a patented material in the mass-concrete work. It would be interesting to learn whether or not the concrete became separated from the steel after a time owing to vibration or shrinkage, thus nullifying the value of that method of waterproofing; this seemed quite possible. That the Wabash Railroad should require the monthly inspection of concrete work showed its conservatism, and was in exact line with Mr. Goodrich's ideas. It was often claimed that because reinforced concrete did not require such constant maintenance as was necessary with steel or timber, therefore less inspection was necessary; but there were so many causes acting to produce unforeseen results in all structures, that the inspection carried out by this railway-company was altogether on the side of good practice. The standard culvert- and pipe-designs adopted by the railway were well worth consideration, although it would seem possible to produce sufficiently satisfactory designs without the use of so much reinforcement.

Dr. Michäelis and Dr. Koenen. Dr. W. Michäelis and Dr. Koenen remarked that there was in general nothing new in the constructional features of the works described by the Authors. The execution of the Sangamon bridge abutments was interesting, but would not appear to possess any special advantages, either on technical or on economic grounds. The reinforcement, looking to the extreme thicknesses of the concrete, would appear to be of excessive strength, and the employment in all cases of steel bars of large sectional area was remarkable. Since the amount of adhesion surface in proportion to the sectional area was comparatively small in bars of large size, it became necessary to increase the holding power of these bars artificially by the use of corrugated surfaces. It appeared to have been for this reason that the relatively costly Johnson bars had been employed. The placing of the steel reinforcement did not in all cases coincide with the practice adopted in Germany. The relatively heavy

Dr. Michäelis and Dr. Koenen.

horizontal reinforcement of the wing- and side-walls was surprising, because these were only employed as ordinary retaining-walls; and so also was the absence of steel reinforcement in the tension-zone of the negative angular stresses (*Eckmomente*) of the four-sided culverts.

Mr. H. St. G. Robinson.

Mr. H. St. G. Robinson observed that while the Paper was interesting as illustrating the application of reinforced concrete to railway underbridges, some of the examples could hardly be classed as reinforced concrete, being principally composed of ordinary steel construction. It was generally assumed that concrete had a very limited application for underbridge construction in Great Britain, but this was really not the case. He would draw attention to the West Highland Railway extension to Mallaig, commenced in 1897. This line, as well as the Highland Railway itself, was particularly rich in examples of economical concrete underbridges built in situ. A comparison of the plain concrete bridges on the Highland Railway with the reinforced-concrete bridges given in the Paper was instructive. No doubt many would be surprised at the abnormal strength of the Sangamon River bridge. It was difficult to see how any appreciable tension could be imposed on the reinforcement, even using the assumed live load. The reinforcement was, of course, useful for compression, but there was sufficient concrete in the ring to take this. How heavy this bridge was would be evident from the following Table showing corresponding dimensions of the centre span of the Morar viaduct, West Highland Railway extension, with the Sangamon River bridge, both being of the solid-ring type.

Bridge.	Span.		Rise.	Thickness of arch at crown.	
	feet	ins.	feet	feet	ins.
Sangamon River	85	10	30	3	9
Morar Viaduct, central arch, 1897. .	90	0	24	3	6

It might be of interest to recall the construction of the Borrodale viaduct.[1] This had a centre span of 127 feet 6 inches and a rise of 22 feet 7 inches; the arch-ring was 4 feet 6 inches in thickness and was of concrete throughout. The parapets were of stone. The total cost of this viaduct was £2,109. For brick, stone, or plain concrete arches of moderate span it was usual to design the arch-

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[1] W. S. Wilson, "Some Concrete Viaducts on the West Highland Railway." Minutes of Proceedings Inst. C.E., vol. clxx, p. 304.

Mr. H. St. G. Robinson.

ring of uniform thickness throughout its width, but for reinforced-concrete arches of moderate or long spans the solid-ring type was certainly not economical. It was usual on the Continent to construct reinforced-concrete arches much after the style of steel arches, the arch-ring consisting of a number of parallel ribs. The ribs were placed under each line and they could thus be readily proportioned for the loads to be carried. The decking was usually carried on reinforced-concrete beams and columns, so that the arch-rings were loaded at definite points and thus approached nearer to the condition usually assumed in calculation. Bridges constructed in this manner were very economical and usually could be constructed for less than the cost of brick or steel bridges. A description of a recently-constructed reinforced-concrete road-bridge designed by him would illustrate this point. This bridge carried a main road over the River Exe at Thorverton. The span was 85 feet in the clear, the total width was 25 feet, and the rise was 10 feet; the four ribs were each 3 feet deep at the abutments, and the total thickness at the crown, including the decking, was 2 feet. The two centre ribs were 18 inches wide and the two outside ones 15 inches. There were cross beams dividing the decking into panels measuring 4 feet 4 inches by 7 feet in the clear, and on account of this construction only a 6-inch decking was required. The cost had been much less than that of a steel girder-bridge or a stone arch designed for the same load and of the same span, being about £1,650. The bridge was of reinforced-concrete throughout, the arch being formed of ribs and the decking carried on beams and columns; the abutments were hollow and were founded on rather soft red sandstone. The structure was tested shortly after completion with traction-engines and stone-trucks, and the deflection was practically imperceptible. The unit stresses allowed in reinforced-concrete structures on the Wabash Railroad seemed fairly high, and he would ask if the unit stresses on p. 134 had been used in the Sangamon River bridge. In a recent article on this bridge,[1] the following statement appeared. "The assumed live load on the structure is 9,000 lbs. per lineal foot of each track. The allowed working-stresses are 600 lbs. per square inch for compression in concrete and 12,000 lbs. per square inch for tension in steel." Would the Authors kindly state if the revised figures on p. 134 were correct. The standard reinforced-concrete box culvert described on p. 141 and in Fig. 10, Plate 5, was of a type much favoured by American engineers. It was difficult, however, to see the object of adopting

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[1] *Engineering News*, vol. 57 (1907), p. 323.

Mr. H. St. G. Robinson.

this type in situations where the headroom was not limited. He was acquainted with several American box culverts under embankments 30 to 40 feet deep. They might be easy to construct and were certainly easy to analyse; but the arch had many advantages over the beam, especially when situated some distance below rail-level, and it was much more economical; the saving amounted in some cases to as much as 25 per cent. in the concrete and 50 per cent. in the reinforcement. It would be interesting if the Authors would give their opinion as to the advantages of the box culvert over the arch in such situations.

Mr. P. J. Robinson.

Mr. P. J. Robinson asked if any tests had been made to determine the ultimate compressive strength of the concrete employed. Also, what was the modulus of elasticity of the high-carbon steel used for the reinforcement? The compressive working-stress on the concrete was somewhat higher than that usually allowed. He also wished to point out that, taking a parabolic distribution of stress in the concrete, the symbol q given in Appendix I was a very different thing from the reciprocal of the factor of safety. Thus, when the stress at the compression surface of the beam was less than the ultimate compressive stress, the locus of the vertex of the parabola was taken by Professor Talbot to be above the compression surface as shown in *Fig. 13.* From the general formula for a parabola it might be shown that

$$c = \frac{c_{ult}}{l^2}(2\,l - y)\,y.$$

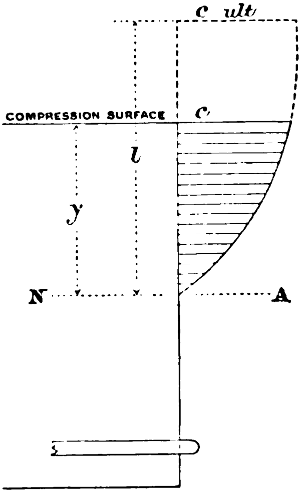
And as the strain diagram for pure bending must be linear

$$\frac{c_{ult}}{l} = \frac{c}{y} \quad \therefore y = q\,l.$$

Substituting for y in the first equation the formula became:—

$\frac{c_{ult}}{c} = \frac{1}{(2-q)\,q}$, where $\frac{c_{ult}}{c}$ was the ratio of ultimate to working compressive stress, or factor of safety. When $q = \frac{1}{4}$, $\frac{c_{ult}}{c} = 2{\cdot}2$, which was a very low factor of safety for the concrete, especially in view of the fact that the factor of safety for the steel was almost 6. The ultimate compressive strength of the concrete would probably

Fig. 13.



Mr. P. J. Robinson. be more than 1,760 lbs. per square inch; it might be inferred, from the sentence, "At one-fourth of the ultimate deformation, which is about the working load," that a factor of safety of 4 had been used, making the ultimate resistance of the concrete about 32,000 lbs. per square inch. With a factor of safety of 4, the assumed distribution of stress was almost linear, and the value of q became 0·134. The results obtained by using this value for q in the parabolic formulas approached very closely to those given by the much simpler linear-distribution formulas. Using the latter, with 1 per cent. of steel fully stressed, the stress at the compression surface worked out at 857 lbs., and the moment of resistance at $0 \cdot 18\ cbd^2 = 0 \cdot 18 \times 800\ bd^2 = 149\ bd^2$. In a Paper[1] recently contributed by him, the parabolic theory of compressive stress distribution was discussed, and formulas were developed in which the factor of safety could be employed directly. This was a more convenient method than that of using the ratio of deformation at working-load to ultimate deformation.

Mr. Wentworth-Sheilds. Mr. F. E. Wentworth-Sheilds remarked that in some of the structures described the concrete itself was of such massive dimensions as to render it very doubtful whether the reinforcement was an economical addition; and it seemed hardly possible that in such structures the engineer had really provided for a stress of 18,000 lbs. per square inch in the steel, and of 800 lbs. per square inch in the concrete, as stated in the Paper. The designs created the impression of having been made by one who had not attained to complete faith in the material he was using, and in the formulas which he employed for ascertaining its strength. Many engineers, both in Great Britain and abroad, had felt some uneasiness in using reinforced concrete for railway-bridges and other heavy structures. The various tests which had now been made on reinforced concrete had established its reliability under the loads which Professor Talbot's and other similar formulas would pronounce to be safe. But in railway work it was still feared that the severe conditions of rolling loads and exposure to wet weather might in time have a destructive effect on reinforced-concrete overbridges. He had had opportunities of examining a good many structures in reinforced concrete, and his experience had convinced him that it was quite possible to design a structure in this material which would outlast any ordinary steel structure which was put to the same uses. But in order that such structures might last it was essential that very great care should be

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[1] "Formulas for the Strength of Reinforced-Concrete Beams." Minutes of Proceedings Inst. C.E., vol. clxxiii, p. 222.

Mr. Wentworth-Sheilds.

taken, and that the designer should have a clear grasp of the conditions necessary for long life. At Southampton, several structures had been erected in reinforced concrete by the London and South Western Railway Company and others. Several of these had stood well, and some, on the other hand, had shown very marked deterioration. Curiously enough, the oldest of all—a jetty built about 11 years ago—had stood best, and had not cost a penny for maintenance since its erection. At that port, therefore, there had been an excellent opportunity of learning under what conditions reinforced concrete was likely to deteriorate, and what precautions should be taken to prevent it from doing so. In this way it had been learned that one of the worst possible conditions tending to rust the steel inside a concrete covering was damp, salt air. At first it was feared that the portions of structures like jetties which were constantly immersed in sea-water would suffer most. But it was found that in fact such portions did not suffer at all, while the portions in the damp air above the water were sometimes severely attacked. Experience had shown, further, that in order to prevent the steel from being attacked under such conditions, the concrete must be specially dense and well-proportioned and that fine aggregate must be used. The Authors stated that for watertight work the concrete on the Wabash Railroad contained no stones at all, but only sand. This was in accord with the best French practice. The Bonna pipes, laid as the main sewers of Paris, some of which were 17 years old, were all made of cement and sand mixed in the ratio of about 1 to 2. Again, it was important for watertight work that the proportion of steel should be low, or rather that the space between the reinforcing bars should be ample, so that each bar might be thoroughly well covered by the concrete. There was a tendency in some of the designs for English work which were imported from France, to cut down the dimensions of beams by increasing the quantity of steel in them. He had seen small rectangular beams with 7 per cent. of reinforcement at top and bottom. This involved crowding the bars, which was not advisable, especially in damp situations. The designers of the Wabash bridges had avoided that fault in many of their structures, though not in all. Another precaution to be observed—perhaps the most important of all—was that no electric currents should be allowed to pass through the substance of the reinforced concrete. It had been clearly proved at Southampton that even very mild currents due to electrical leakage aggravated the tendency of the steel to rust in moist air. Regarding deformed bars, it did not appear that their advocates had established any case for them. Well-designed beams

Mr. Went-worth-Sheilds. with smooth bars seemed to have given every satisfaction. But whether smooth or deformed bars were used, it seemed to him that round bars were better than square ones, as it was possible to bed them more effectually into the concrete, and thus obtain closer contact. This meant not only better adhesion, but also increased protection to the bar itself against possible rusting.

Mr. Sims. Mr. **Thomas Sims** noticed that the use of reinforced concrete did not appear to have extended to railway-sleepers on the Wabash Railroad. He had hoped that the Authors would have made some allusion to that aspect of the question, because although reinforced-concrete sleepers had been, to some extent, used elsewhere, he understood that their use was at present somewhat of an experiment in consequence of the possibility of their being injured by the excessive vibration caused by passing loads. Perhaps in replying the Authors might be willing to give any information they could on this matter. Reinforced concrete had now many advocates and many critics, but the statement that no special loads were used to test the structures was some evidence of the great confidence which was placed in this comparatively new method of construction on the other side of the Atlantic.

Mr. Starbuck. Mr. J. E. **Starbuck**, of the Vandalia Railroad Company, St. Louis, remarked that the assumed dead loads given on p. 133 were the same as used by that company. The live load assumed by the Wabash Railroad gave somewhat smaller stresses than the Vandalia's assumed live load of 5,000 lbs. per lineal foot, plus a concentrated weight of 60,000 lbs. so placed as to produce the greatest stress in the structure. Instead of the impact formula used by the Wabash Railroad, he had used a percentage of the concentrated load, varying with the depth of the superimposed embankment, which gave results not differing greatly from those given by the Wabash formula. The greatest difference between Wabash Railroad and Vandalia Railroad practice lay in the stresses allowed for steel in tension and concrete in compression, the former being 18,000 lbs. per square inch tension in steel, and the latter one-fifth of the elastic limit. For the Johnson corrugated bar, which was the only bar which had been used hitherto by the Vandalia Railroad, the elastic limit was assumed at 50,000 lbs., making the working-stress 10,000 lbs. The Wabash Railroad allowed in 1 : 2 : 4 concrete 800 lbs. per square inch compression in cross bending, and 600 lbs. per square inch in direct compression. The Vandalia Railroad allowed one-fifth of the ultimate strength of the concrete, which was assumed to be 2,700 lbs., in both cross bending and direct compression, making the working-stress 540 lbs. They considered structures of reinforced concrete as being permanent, and

Mr. Starbuck.

by the use of these low unit stresses provided for any probable increase in the weight of rolling stock. It was their practice to use a wet mixture for reinforced concrete, the reasons for so doing being those stated in the Paper. He also agreed with the Authors that it was best not to rely on plain bars for reinforcement in concrete, and that a deformed bar of some sort should be used. The Wabash Railroad had had a somewhat longer experience in the use of reinforced concrete than the Vandalia Railroad, and had built a greater variety of structures. On the latter railroad they had built no hollow, counterforted abutments, with thin reinforced face-walls, of the type shown in Fig. 3, Plate 5, but had built two arches of about 20 feet span, having counterforted abutments of plain concrete, with heavy face-walls, and with counterforts 2 feet thick, spaced 2 feet apart in the clear, which had given satisfaction, and had shown some economy as compared with solid abutments. Many culverts of the general design shown in Figs. 10 and 11, Plate 5, had been built on the Vandalia Railroad. The earlier designs had the top slab reinforced on the lower side only, and the bottom slab on the upper side only, as seemed to be the practice of the Wabash Railroad. The latest designs, however, had additional reinforcement on the upper side of the top slab and on the lower side of the bottom slab, at the supports or side walls, this reinforcement extending to points a little beyond the quarter points of the spans, and being carried down into the side walls, on the outer side, to a little beyond the quarter points of the spans of those walls. This change in the method of reinforcement had been made not because of any failures of culverts with reinforcement on one side only of the slab but because it seemed to be the more rational method. Mr. Starbuck had no doubt that reverse stresses were developed at the supports. The culverts were buried under embankments, and there was no way of observing the outer sides of the corners to see if the reversed stresses had caused cracks. In his opinion cracks would be found should the culverts reinforced only on the inner side of the slabs ever be uncovered. If the reinforced-concrete slab were really an elastic beam, then when the ends were absolutely fixed in a rigid support the bending-moment at the support for a concentrated load at the centre would be as great, and for a uniformly distributed load twice as great, as the bending-moment at the centre of the beam, but of the opposite sign. In the box culvert the supports were not perfectly rigid, but so nearly so that for a combination of concentrated and uniformly distributed loads the bending-moment at the supports would still be much greater than at

Mr. Starbuck. the centre, and still of the opposite sign. It seemed to him hardly rational to use no reinforcement at the point where the greatest bending-moment occurred. How much allowance should be made on account of the lack of rigidity of the supports in proportioning the reinforcement between the upper and lower sides of the beam was a question which each designer must answer for himself; but it would seem to admit of little question that some reinforcement should be put in over the supports, and if this were done, the reinforcement of the lower side of the beam at the centre might be reduced. At the beginning of 1910 complete descriptions were published[1] of some typical reinforced-concrete structures built by the Vandalia Railroad, with illustrations. The box culvert shown in Figs. 16 to 21 of the article referred to, had reinforcing bars in the outside of the slabs over and under the centre wall, but no bars at the outside corners of the culvert. Were this culvert being built now, more reinforcement would be used over the centre wall, outside corner reinforcement would be put in, and the reinforcement of the inner sides of the slabs would be somewhat reduced.

Mr. Vawdrey. Mr. R. W. VAWDREY remarked that, while it was of course very desirable that all progress made should be brought before the notice of The Institution by the reading of Papers such as the one under discussion, it appeared regrettable that the bridges, walls, etc., described, should be of such poor design when considered as reinforced-concrete structures. Not a single piece of work described could really be considered to be reinforced concrete, being either of steel with a little concrete protective covering, or of concrete with a little steel reinforcement thrown in as an additional safeguard. No doubt the general American practice leaned in the direction of much heavier design than obtained in England, and it was clear that the question of cost did not arise there to the same extent as it did here. For it was impossible to believe that any of the concrete structures described would have run the gauntlet of the severe competition with other materials and competitive designs which every reinforced-concrete design had to submit to in Great Britain. In this connection it was desirable to call attention to the very serious danger arising from the form of competition which was in vogue in England at the present time with regard to reinforced-concrete work. Practically all such work was designed at the present time by a few specialist firms controlling special systems or forms of reinforcement, which might or might not be patented. This was

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[1] *Engineering*, vol. lxxxix (1910), p. 39.

Mr. Vawdrey.

probably a necessary evil, owing to the very special experience required for the design of such work. It was impossible for the general engineer to have a sufficient amount of such experience unless he was continually engaged in the design of reinforced concrete, which was unlikely to be the case. The great danger, however, lay in the fact that where it was proposed to construct any work in reinforced concrete, a large number of specialist firms were usually asked to prepare designs, and at the same time to give or obtain tenders for their construction. There was some advantage in the submission of designs to competition, although as a rule, this meant a waste of the large amount of money expended in their preparation; but to decide the merits of the competitive designs merely by comparing the prices, which a large number of different contractors had put in upon them, was radically unsound. He knew it to be a fact that in the great majority of cases no attention was paid, in deciding upon the designs, to any consideration except that of price. This was often due to the inability of an engineer inexperienced in reinforced concrete to form in any other way an opinion on the merits of designs before him, but the result was that a premium was put on insufficient and insecure designs, and subsequently on bad workmanship in their execution. The remedy would appear to be to separate the competition in design from that in price. It should be an invariable rule, when the tenders of various contractors were being considered in competition, that they should all be based on the same design and quantities, which should have been definitely adopted before tenders were called in. This was, of course, the case in the ordinary form of competitive tender for general engineering work other than reinforced concrete. The Authors advocated the use of reinforcement having a mechanical bond with the concrete. It was of course essential that the bond between the two materials should be efficient, and absolutely permanent. It was no doubt a fact that with a very high class of materials (that was, a rich mixture of concrete) and of workmanship, a sufficiently good bond could be obtained, at any rate at first, between smooth bars and the concrete. The use, however, of reinforcement with a mechanical bond did not add appreciably to the cost of the structure, and certainly eliminated one element of risk, namely, that of insufficient bond due either to poor workmanship, to deterioration of the bond through lapse of time, or to the subjection of the structure to vibration. It was stated that the corrugated bars used by the Wabash Railroad (these being what were known as "indented" bars in England) were rolled of high-carbon steel, with an elastic

Mr. Vawdrey. limit of 66,000 lbs. per square inch. This appeared to be rather too high; but there was no danger in the use of steel reinforcement having an elastic limit of anything up to 50,000 lbs. per square inch, so long as the working-stress did not exceed half the elastic limit. It was fallacious to argue that the use of a higher stress than, say, 15,000 lbs. per square inch in the reinforcement was liable to cause cracks in the concrete. For if the stress in the steel were limited to an amount which would not cause the concrete to extend more than it was able to do without cracking, it would mean that this stress would be only about fifteen times the tensional strength of the concrete. The latter being, say, 300 or 400 lbs. per square inch, that would give only about 2 tons per square inch on the steel, which would obviously be absurd. The true function of the reinforcement in this connection was to distribute the extension which must necessarily take place in the concrete over the whole length of the beam or other member of the structure under consideration. While the bond between the steel and the concrete remained perfect, the distribution of extension must of necessity take place, and no crack could possibly occur of sufficient size to admit moisture or cause the slightest deterioration in the steel; nor could any crack become visible to the naked eye, while the bond was maintained. It was of course obvious that the higher the stress might be in a reinforcing bar, the greater became the necessity for some form of mechanical bond. One of the chief objections to the use in ordinary steel construction of steel of a higher elastic limit than, say, 30,000 lbs. per square inch, was its unsuitability for drilling, punching, riveting, etc. This objection did not arise in the case of reinforcement, and if bars with a mechanical bond were used, the necessity for splitting or cranking the ends was of course avoided. In such cases, therefore, there appeared to be no reasonable objection to the use of a higher class of steel than that ordinarily employed for steelwork structures. The Authors also raised the question of the best composition for reinforced concrete. Undoubtedly denseness was a very important quality. This must be obtained by very careful mixing and ramming. In proportioning the cement, sand, and gravel or stone, the desired composition of the mortar should first be decided upon, say 1 of cement to 2 of sand, and sufficient of this mortar should be used to completely fill the interstices of the coarse aggregate. The volume of these interstices would depend far more than was generally considered upon the amount of ramming which was used in placing the concrete. To take an extreme case, if the coarse aggregate consisted of a

Mr. Vawdrey.

number of equal and regular cubes, they could be packed together, so that there were practically no interstices at all, and consequently a minimum of mortar would be required. The more nearly such a condition could be approached, the more dense would be the concrete, and the more economical would be its cost, owing to the saving of the cement and sand. Obviously the best class of aggregate in reinforced concrete was that which would best, as it were, fit together, and reduce the percentages of interstices to a minimum. This was most readily done when the size of the portions varied as much as possible. With regard to the possibility of making concrete waterproof, among a considerable number of concrete tanks, water-towers, etc., which he had constructed recently, might be mentioned a reinforced-concrete water-tower 70 feet high, holding 50,000 gallons of water. The depth of the water in the tank was 12 feet, and the thickness of the sides and floor of the tank was 6 inches. The aggregate was of crushed granite varying from $\frac{3}{4}$ inch to $\frac{1}{4}$ inch, and the concrete was in the proportion of 1 : 2 : 4. No rendering, patent waterproof cement, or other special precautions were used, but the concrete was, of course, carefully mixed, and well rammed into position. The result had been eminently satisfactory, the tank being perfectly watertight from the very day of its completion. This appeared to indicate that much of the rendering, bituminous sheeting, etc., often used, was entirely unnecessary if proper care were taken in proportioning the concrete, and still more in placing it.

Prof. Warren.

Professor W. H. **Warren** considered that the formula $I = \frac{L^2}{D + L}$ expressed the effect of the impact of rolling loads in a satisfactory manner, but suggested that in view of the relatively small experience in the behaviour of reinforced concrete in structures subjected to the vibratory loads produced by heavy locomotives, and to the known reduction in strength produced by repetitive stresses especially when accompanied by shock, the formula should for the present be written

$$I = 1 \cdot 25 \frac{L^2}{D + L}$$

The use of the standard loading adopted in the Paper, in preference to equivalent uniformly distributed loads was to be commended. These standards had been suggested by Mr. Theodore Cooper in his well-known specifications for the design of railway-bridges; and various simple methods had been developed for the determination of the position of the wheel concentration for producing the maximum shears and moments, as well as the stresses for this position of the

Prof. Warren. loading. In regard to the safe working-stresses given in the Paper for structural steel and concrete, he considered that 600 lbs. per square inch for concrete subjected to direct compression, and 800 lbs. per square inch extreme fibre-stress for compression in beams were not justified by tests of 1 : 2 : 4 concrete, or by the Summary of Building Regulations given in Appendix II. If these working-stresses were intended to apply to concrete beams reinforced on both the tension and the compression side, and to columns reinforced with longitudinal rods, with spiral hoops, or with transverse ties, they might be considered safe, depending on the nature and percentage of the reinforcement; but as printed in the Paper they appeared to apply to plain concrete. The following considerations showed, in his opinion, that the working-stresses printed in the Paper were too high. It was well known that the strength of a cube of concrete was greater than that of a prism whose height exceeded twice the base, and failure occurred by shearing along a plane inclined to the axis at an acute angle. The average strength of concrete cubes of 12 inches side, 18 to 24 months old, and made of a wet mixture, was given in the Paper as 2,180 lbs. per square inch, which agreed fairly well with the results of numerous tests made by him, although higher results had been obtained in Europe. If such concrete was used in columns, reinforced by 0·8 per cent. of steel longitudinal rods, and with cross ties spaced at a distance apart slightly less than the diameter of the column, the strength of the column might be considered to be approximately equal to the cubic strength of the plain concrete. If the length exceeded sixteen times the least lateral dimension, the column must be calculated with regard to buckling. The regulations of the Royal Prussian Ministry of Public Works, 1907, gave the working-stress for columns as $\frac{1}{10}$ of their ultimate strength, and for buckling $\frac{1}{5}$ of their ultimate resistance to buckling. In beams the safe compressive stress was given as $\frac{1}{6}$ of the ultimate strength of the concrete, and the safe stress in the steel reinforcement as 14,223 lbs. per square inch. Again the "Leitsätze" allowed a stress of 498 lbs. per square inch as safe for concrete columns, provided that the cubical strength of the concrete at 28 days was 2,560 to 2,845 lbs. per square inch, which was much stronger than the wet mixture mentioned by the Authors. The extreme fibre-stress in beams was allowed to reach 569 lbs. per square inch with such concrete. The strength of reinforced-concrete columns in which the length did not exceed twelve times the diameter, reinforced with longitudinal rods and cross ties spaced at a distance apart slightly less than the diameter of the column, according to experiments by Mr. Considère and others, including Professor Warren,[1] might be expressed thus Prof. Warren.

$$P = \sigma_c (A + na)$$

where $n = 10$ to 15 depending on the concrete, and σ_c denoted the ultimate strength of the unreinforced concrete, a being the area of the longitudinal rods. P then denoted the ultimate load carried. If spirals were used, combined with about the same weight of longitudinal rods,

$$P = \sigma_c A + \sigma_e (a + 2{\cdot}4\, a_1)$$

σ_e denoting the limit of proportionality of the stresses and strains in the steel, and a_1 the area of imaginary longitudinal rods of the same weight as the spirals in an equal length of column. In regard to the use of deformed bars, which was characteristic of American practice, in preference to smooth bars as mainly used in European practice, the advocates of the former claimed greater security, due to the mechanical bond provided by the deformed reinforcing bars, which justified their higher cost; while advocates of the latter considered the ordinary adhesion between the smooth bars and the concrete to be quite sufficient in a properly-designed reinforced-concrete beam. He considered that plain smooth rods, formed at the ends with a hook, bent to a semicircle, having a diameter five times that of the rod, should be used, with no sharp bends where the horizontal reinforcement was turned up in order to deal with the shearing-stresses. This arrangement was recommended by Considère, and provided considerable frictional resistance without causing fracture in the concrete. It was worth while noting that the mechanical bond with the deformed bars could be effective only when the reinforcement was anchored in a relatively large mass of concrete. In regard to the use of a wet mixture, it appeared desirable not to use more water than was necessary to produce such a consistency of the concrete as would enable it to cover the reinforcement completely with ordinary tamping. Such concrete usually contained about 50 per cent. more water than earth moist concrete. In Appendix I the Authors quoted Professor Talbot's formulas derived from experiments on reinforced-concrete beams tested to destruction. Under these conditions the form of the curve representing the distribution of compressive stress between the extreme layer and the neutral axis was parabolic, whereas under safe working-conditions the parabola was flat enough to be

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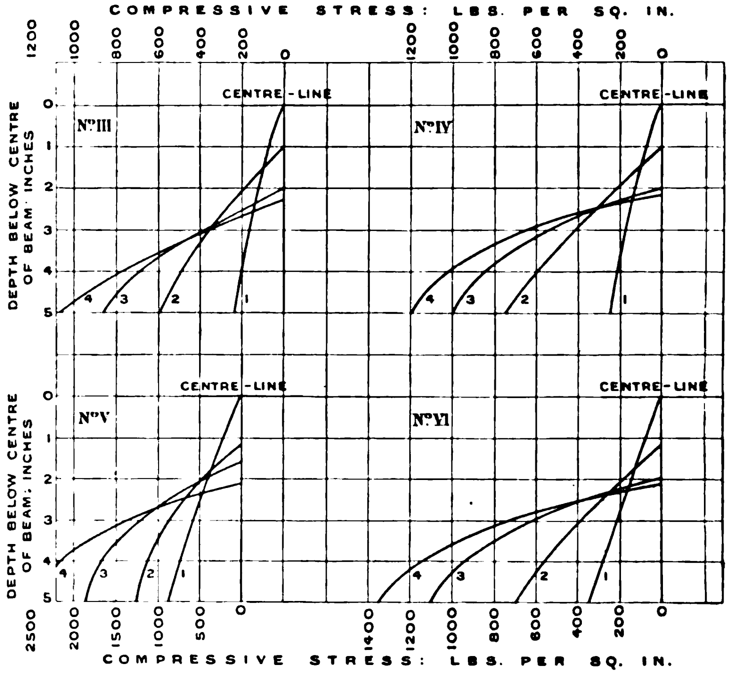
[1] "Further Experiments on the Strength and Elasticity of Reinforced Concrete." Journal and Proceedings of the Royal Society of New South Wales, vol. xxxviii (1904), p. 140.

Prof. Warren. represented with sufficient accuracy by a straight line. *Fig. 14* showed some of the results[1] obtained by him by means of observations taken at the same time on each side of a beam with eight sets of Martens mirror apparatus—four sectors and two dials. The more rational method of designing a reinforced-concrete beam appeared to him to be that in which the required bending-moment and safe working-stresses were used in formulas derived from the stress-strain curves agreeing with these stresses. Formulas expressing the conditions at rupture were not rendered applicable to working-conditions by simply dividing the results by a factor of safety. While agreeing with the Authors in the advantages of reinforced concrete, even in structures subjected to rolling loads as in railway-traffic, he did not agree with the

Fig. 14.



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[1] "Reinforced Concrete, Paper III"; Journal and Proceedings of the Royal Society of New South Wales, vol. xxxix (1905), p. 49. *See also* W. C. Popplewell, "Experiments on the Strength Properties of Reinforced-Concrete Beams"; Minutes of Proceedings Inst. C.E., vol. clxxvii, p. 177.

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high working-stresses specified for the concrete, either in direct compression, or in the extreme fibre layer of a beam. He also differed in regard to the use of a stress of 18,000 lbs. per square inch in the steel reinforcement, and considered that it could not increase the economy in the design unless the cost of the steel was more than 100 times that of an equal volume of concrete. Professor Merriman had shown that:—

$$\frac{\sigma_s}{\sigma_c} = r = 1\cdot17\sqrt{np}$$

where n denoted the ratio E_s/E_c and p was the ratio of the cost of the steel to the cost of an equal volume of concrete. The application of reinforced concrete on the Wabash Railroad, as illustrated in the Paper, was of great interest in so far as it applied to structures subjected to railway-traffic.

Lieut.-Colonel Winn, R.E.

Lieut.-Colonel J. Winn, R.E., considered that the success which had attended the employment of reinforced concrete for bridges, culverts, etc., on this railway would tend to remove the fear often entertained that this combination of materials might not be able to withstand the excessive vibrations and shocks to which such structures were exposed. It should, moreover, encourage the use of what was undoubtedly an economical form of construction, and justify the extra cost of careful supervision during construction, upon which depended the success of the whole. With reference to the reinforcement, the Authors seemed to advocate deformed bars as essential, on the ground that proper adhesion of the concrete could not take place otherwise; which was another way of stating that adequate frictional resistance between the concrete and reinforcement could not be obtained by the use of plain bars. In view of the fact that numerous structures on the Continent, which were equally as successful as those in America, had been made with plain bars, it did not seem that this contention should go unchallenged, especially as it might dissuade engineers from adopting reinforced concrete for railway work in many localities where special deformed bars were not readily obtainable. With such high tensile stress in the steel as 18,000 lbs. per square inch, it perhaps might be conceded that special deformed bars were desirable; but at the usual stress allowable for mild steel, of 15,000 lbs. per square inch, and with such a low allowable frictional resistance as even 50 lbs. per square inch, it would be found in practice that the use of special bars was not essential. Plain rods therefore, might be used with confidence, and there were many occasions when these were obtainable, and special bars were not. As to the concrete being put in wet, "poured" rather than "placed," Lieut.-Colonel Winn, R.E. it was of the greatest importance that this method of construction was now recognized as sound. Mr. Rafter's experiments had demonstrated that though a dry mixture gave the best results, still a wet mixture produced a concrete amply strong in view of the low stresses which were taken as the basis for calculating the members. It was true that these wet mixtures could not be rammed, and ramming was desirable for consolidation as a rule, but they could be and should be worked vigorously with a sharp instrument (such as the blade of a trowel) as soon as deposited, and this ensured an intimate contact with the reinforcement which was an essential condition of success. The resulting frictional resistance, as the concrete set, was consequently enhanced, and in view of the fact that the concrete itself was amply strong in compression, the slight loss in comparison with a drier mixture was more than compensated for by the improved grip, due to this greater intimacy of contact. It might be argued that the use of deformed bars ensured a better grip, and this was readily conceded, but it seemed desirable to point out, in view of the contention of the Authors, that the grip of the concrete when properly worked round the bars was adequate to the requirements of the case.

Mr. Cunningham. Mr. CUNNINGHAM, in reply to the Discussion and Correspondence, wished it to be understood that the illustrations in the Paper, showing the use of reinforced concrete on the Wabash Railroad, were not presented with the idea that they were ideal, but to emphasize the fact that structures of this kind had been employed under heavy railway-traffic, and that their behaviour during the time that they had been in service showed that reinforced concrete, if properly designed, could be used with impunity for carrying such traffic. At the same time, while the criticisms seemed to be severe, no apologies were offered for the designs, because, though most of them were made at a time when reinforced concrete was little understood, and when very few practical tests had yet been made, reinforced-concrete structures for railway work in the United States followed, even to-day, very closely the designs shown in the Paper. The plans submitted with the Paper (Plate 5) were intended to give an idea of the variety of designs that had been used; and, judging by the criticisms, it was evident that the reasons for the different structures had not been properly understood; he would therefore explain each design illustrated, and hope in this way to reply effectively to the criticisms.

The Sangamon River bridge (Fig. 2) had been particularly criticized on account of the thickness of the arch-ring. It was claimed that, because structures of about the dimensions of this

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bridge had been built in England with arch-rings of approximately the same thickness, no saving had been made by using concrete and reinforced concrete. It must be remembered that engine- and train-loads in the United States were more than double those on English roads, and therefore necessitated much heavier sections than those required by English train-loads. It was the practice among American engineers to design reinforced-concrete arches for railway-bridges so that the stresses due to the live and dead loads lay within the middle third of the arch-ring, and the steel was placed in the arch-ring only to take care of the temperature-stresses. It should be remembered that the variation in temperature where this bridge was situated was far greater than in England, consequently it was very necessary to guard against the effect of temperature, and to provide steel near the extrados and intrados of the arch-ring to take care of the tensional stresses due to changes of temperature. In the design of the arch-ring it was ascertained that, with a temperature-change of 40° F., a tension of 240 lbs. per square inch in the concrete was found at the abutments and of 100 lbs. per square inch at the crown; so that, unless steel had been introduced to take these tensional stresses, the concrete would have been overstressed in compression. The Sangamon River bridge had been compared with the Wharncliffe viaduct in England, though that viaduct had a clear span of only 65 feet, whereas the Wabash bridge had a clear span of practically 86 feet; moreover, as the Wabash bridge was built on a skew of 45°, the comparison was unreasonable. It was unnecessary to call attention to the ambiguous stresses in an arch-ring of such narrow width compared with the span, when the abutments and piers were at an angle of 45° with the axis of the arch-ring, as the lines of thrust must be diverted in a lateral direction in order to be resisted by the abutments; and for this reason alone, the number, size, and position of the bars as used were justifiable. This being the only arched bridge in existence with such a large skew, it had been absolutely necessary to be very conservative in the design of the arch-ring, and it had been deemed advisable to keep the allowable stresses low; consequently, they were not permitted to exceed 600 lbs. per square inch in the concrete and 12,000 lbs. per square inch in the steel. Particular features that should have been considered in this bridge were the methods used in taking care of the load on the foundations by using reinforced-concrete slabs and in the design of the abutments. Notwithstanding the criticisms of the abutments, there was not in existence to-day a structure of such height built with as little

Mr. Cunningham. material as in these abutments. The side walls were practically plain concrete 3 feet in thickness, while the vertical wall dividing the abutment into two sections was of the same thickness and had a very small amount of reinforcement in it, to take care of the tensional stresses due to lateral play. The lower horizontal wall had only sufficient reinforcement in it to provide for its own weight, and was placed in that position to tie the high vertical wall at the centre, while the top slab carrying the track was sufficiently reinforced to take the tensional stresses in a beam 25 feet from centre to centre of the supports, as this was the true theoretical span. One speaker had remarked that this slab could just as readily have been an arch, but had failed to take into consideration the thrust of the arch. Had arched construction been used to carry the track on the abutments, it would have been necessary either to build very much heavier side walls or to place a large amount of reinforcement near the inner edge of the vertical walls to take care of the arch-thrust, and both of these plans would have been more costly.

Passing to Fig. 3, which represented reinforced-concrete abutments of a type in which the weight of the earth backing was utilized to prevent overturning, several abutments of this style had been constructed on the Wabash Railroad, and the first was built of thinner sections than the one shown; but it was found that the cost of placing the concrete and steel was more than it should have been, owing to the small spaces in which to work. It was therefore decided in the future to construct them with sections not less than 18 inches in thickness and it had since been found that the later abutments were, on this account, cheaper to construct. Even using sections of this size, the saving on large abutments varied between 10 per cent. and 50 per cent., depending upon their size and situation. On the Wabash Railroad material was brought to the site and unloaded from trains, good gravel and sand were obtained on the line, and cement was manufactured close to it; and under these conditions the saving was at least 10 per cent.: whereas, if the materials were brought long distances and transported from trains to the site in wagons, the saving might be as much as 50 per cent. As an interesting illustration of this, the New York, Ontario and Western Railway built two abutments for a bridge, one of which was of reinforced and the other of plain concrete. The reinforced-concrete abutment cost 25 per cent. less than the plain-concrete, though the abutments were built simultaneously and under the same conditions.

With regard to the different designs of reinforced-concrete floors to carry ballasted track, Figs. 4 showed a design which was generally

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used where the distance between the base of the rail and the underside of the bridge was not important; the designs in Figs. 6 and 7, however, might be used equally as well in such situations, but that of Figs. 5, with a trough-shaped floor, was used only where the distance between the base of the rail and the underside of the girder was an important feature. In Figs. 4, the slab supported by the **I** beams at intervals of 18 inches acted as a continuous beam, and for that reason it must be reinforced longitudinally at the top and bottom. It must be remembered that the bridge deflected as a train passed over it, carrying with it the concrete slab, which therefore required additional longitudinal rods in the bottom to take care of this deflection; and further, that the cross beams carrying the slab also deflected on account of the heavy engine-loads, for which reason rods cross-wise of the slab and parallel to these beams were placed in the bottom of the slab, to take care of the deflection in this direction. It had been demonstrated practically that all the rods shown in the design were necessary, otherwise cracks would develop either longitudinally or cross-wise of the slab.

The Junction Avenue bridge built at Detroit, referred to in the Paper but not illustrated, was of similar construction to that shown in Figs. 5, and the distance between the base of the rail and the underside of the girder was 1 foot 9 inches. It was of course very easy to construct a bridge-floor of less depth between the base of the rail and the underside of the girder, provided steel decking was used, and the sleepers were laid directly on the steel plates, with ballast between them only. If no ballast were used, the rail could be attached to the cross beams directly, and the distance need then be only 12 inches, even with the locomotive-loads used in the United States. In American cities, however, steel decking was not allowed unless sufficient ballast was placed on it to absorb the noise from passing trains; consequently engineers in America, when they used such construction, covered the deck with concrete and allowed ballast between it and the underside of the sleeper. The reason, then, for encasing the **I** beams with concrete, as shown in Figs. 5, was one of necessity and not of choice, though it had been found that floors constructed in this manner were very rigid and sound-absorbing. It was evident that in order to get a shallow ballasted floor, it was necessary to place the concrete slab on the bottom flange of the **I** beam, and to prevent the ballast from coming in contact with the beams it was essential that they should be covered with concrete. In order that the extra concrete around the beams should be self-supporting, rods were placed in it, close to and parallel with the bottom flange of the beams. The rods tying

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the slab together so as to prevent it from cracking transversely were threaded through holes in the web of the I beams close to the bottom flange.

Figs. 8 showed what was called in the United States a reinforced-concrete trestle. The abutments were not reinforced, because this was not necessary, but the piers were reinforced to take care of the tensional stresses due to the eccentricity of the live load, and to prevent them from being broken by drift and ice, while the slabs carrying the track were reinforced sufficiently to carry the heavy engine-loads coming on them. This kind of structure was used in situations where there was no liability to scour.

Fig. 9 showed a structure originally designed to utilize old stringers removed from light bridges. Only a few of these bridges had been constructed, as it had been found that it was cheaper to sell the old girders and use reinforced-concrete slabs than to build bridges of this kind, and this design was no longer used. These bridges had proved to be very substantial, and none of them had required any repairs, though it should be noted that the sections of reinforced concrete were comparatively thin.

Figs. 10 and 11 showed a reinforced-concrete box used in places where the foundation was soft and liable to scour. It had been found, not only on this railway but also on several others, that a culvert of this description, while costing no more than a plain concrete arch, had the advantage that it provided a better waterway opening, and withstood settlement better. With plain concrete arches, it was necessary to have an absolutely solid foundation, otherwise cracks would develop and it had been found on the Wabash Railroad that it was almost impossible to construct, under a heavy embankment, a plain concrete arch that would not crack owing to settlement in the foundation. For this reason alone it had been found advisable to use a flat-topped reinforced-concrete culvert, and, as stated previously, the cost was no more. Reinforced-concrete arches could undoubtedly be used to advantage and with economy on many railways, but, on account of the cheapness of the materials of concrete on the Wabash Railroad, reinforced-concrete arches involving thin sections of concrete, a large number of rods, and extra labour, were less economical than the heavier reinforced-concrete boxes.

Figs. 12 showed reinforced-concrete pipes with a 4-inch ring. From the Figure the pipe would appear to be very heavily reinforced, but it would be noticed that $\frac{1}{4}$-inch longitudinal bars were used, with wire fencing of small gauge, and even with the reinforcing shown, it had been found that under very high banks, a great

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deal of distortion occurred in the pipes, which would indicate that they would probably be better if the reinforcing were heavier.

Concrete sleepers had not been tried to any large extent on the Wabash Railroad, because it was felt that reinforced concrete in such small masses would soon be battered to pieces under the heavy impact due to the engine wheel-loads in the United States. A few reinforced-concrete sleepers had been laid on a piece of line near St. Louis, but these had soon been broken to pieces under the heavy traffic. Concrete sleepers of all manner of design had been tried on several other railways, but so far none of them had been able to withstand the heavy traffic. In all probability reinforced-concrete sleepers might be used satisfactorily on European railways because of the lighter locomotives used there.

There was a great difference of opinion as to whether there was any necessity for the use of deformed instead of plain bars in reinforced concrete, though it was evident that a deformed bar transmitted the stresses into the concrete in increments of its entire length, whereas the stresses were not so distributed by plain bars, because the concrete was liable not to grip the bar uniformly along its whole length, and there were many places in the concrete where, owing to poor workmanship, there was no grip at all on the bar. The difference between the transmission of stresses into the concrete by a deformed bar and a plain one was only one of magnitude, as a plain bar transmitted the stress owing to the minute unevennesses in the bar; while in the deformed bar these unevennesses were exaggerated, and so became properly effective. It had been shown by experiment that plain bars lost a great deal of their adhesion if the concrete became wet, so that it was necessary in such places to use deformed bars. Many foolish attempts were made to anchor plain bars in concrete, which resulted in very expensive reinforcement, and the end sought was not accomplished because the bars so anchored did not transfer the stresses through their entire length to the concrete as they should, but simply to the ends of the bars, and thus acted as truss-rods. In railway-structures it was generally very necessary to use high-carbon steel bars in order that high unit stresses might be used, otherwise the large stresses due to the heavy locomotive-loads would require a double spacing of bars, as a single spacing would not permit the bars to be placed at proper distances apart to allow sufficient concrete between them, on account of the number of bars required. It had been shown by experiment that plain bars of high elastic limit would slip in the concrete under comparatively small stresses, and it was generally agreed, even by advocates of plain bars, that

Mr. Cunningham. high-carbon steel should not be used, for this reason. Consequently, when high-carbon bars were used it was necessary that they should be deformed. Plain bars in the United States were made of bridge-steel, which had an elastic limit not exceeding 36,000 lbs. per square inch, and one-third of this value, or 12,000 lbs., might be used as an allowable unit stress in reinforced-concrete structures. The high-carbon steel bars used for reinforced concrete on the Wabash Railroad had an elastic limit of 54,000 lbs. per square inch, so that 18,000 lbs., or one-third of this amount, was the proper value to use. This meant, then, that two high-carbon bars were of equal value in point of strength to three low-carbon bars of the same area; in other words, a saving of one-third was made in the quantity of steel used if high-carbon bars were used, so that when plain low-carbon or bridge-steel bars cost $30·00 per ton, an extra amount of $10·00 per ton might be paid for the high-carbon deformed bar to produce the same strength due to the steel in reinforced concrete.

Objections were made to high-carbon steel by those in favour of the bridge-steel because high-carbon steel was never advocated for bridges; but the fact was lost sight of that in steel bridges the sections of the steel were large and received severe treatment in punching, riveting, etc., whereas bars in reinforced concrete were of small cross-sectional areas, and therefore less liable to imperfections, and they never received the harsh treatment to which steel was subjected in constructing a bridge. Further, the steel in a bridge received direct shock from the rolling load, whereas in reinforced concrete the shock was absorbed by a large mass of concrete before reaching the steel bars. The best argument that could be advanced in favour of the high-carbon steel bars was that, during all the years that they had been used on the Wabash Railroad, no breakages had occurred in them.

In America concrete had passed the experimental stage, and it had been found in practice that concrete of a wet mixture was much superior to that of a dry, both in strength and in economy. It was formerly the custom to build structures with concrete of a dry mixture and to thoroughly tamp the concrete so as to bring the moisture to the surface before the next layer of concrete was applied, because in laboratory tests it had been found that concrete made in this way showed greater strength than concrete made with a wet mixture, though as time progressed, the tests showed that the concrete made with a wet mixture gained in strength more rapidly than that made with a dry mixture. It must be remembered, however, that laboratory tests often yielded results different from those obtained in practice, and that while a small quantity of concrete could be made with a dry mixture and show better results than with a wet mixture, it did not follow that in large work the result was the same, because the same amount of care in tamping the dry mixture was not exercised in practice. Evidence of this was afforded by the fact that concrete structures built when the dry method was in vogue could not compare in looks and in strength with later structures built with a wet mixture. It had been found by tests that concrete properly made with a wet mixture of 1 part of cement, 2 parts of sand, and 4 parts of gravel, would develop an ultimate crushing resistance of 2,400 lbs. per square inch after the concrete had set and dried for 60 days, so that it would be proper to allow one-third of this value, or 800 lbs., as the unit working-stress in compression for the concrete in reinforced-concrete structures when the proportions for the concrete were as just given. Repeated tests had shown that the fibre-stress in compression could safely be made much larger in beams than in columns, because of the small area affected in beams. Experiments had shown that to obtain failure by compression in beams very high percentages of steel must be used—much higher than were necessary to develop theoretically the average crushing-resistance of concrete blocks of the same mixture. Mr. Cunningham.

Most of the criticisms that had been offered were due to ignorance of the difference between the conditions found in the United States and in Europe. The fact that a thin section of concrete and an abundance of steel was more economical in one locality was no reason why it should be so in another. The materials that entered into concrete were generally cheaper in America than in Europe, whereas labour was much more costly in the former than in the latter, and further, labour was much more effective in Europe than in the United States, and better workmanship could be obtained in the former than in the latter. These causes alone would result in designs in the United States being of heavier sections than those in Europe, because with small sections it was necessary to provide and place properly a large quantity of steel and to work well-made concrete very carefully around the steel; this could undoubtedly be done with satisfaction where the workmen thoroughly understood the work and had been educated up to it. In the United States it was very often necessary to use, for mixing and placing concrete for railway-structures, common labourers who had never had any experience whatever, and until a time arrived when proper labour could be supplied it would necessarily follow that less steel would be used in reinforced-concrete railway-structures in the United States than was customary in Europe.

The state of the art of building in reinforced concrete was just as far advanced in the United States as it was in Europe, but such

Mr. Cunningham. absurdly thin sections as were used in Europe were not favoured in the United States. Arches for highway-bridges and buildings might be built of thin sections of reinforced concrete, because they did not have to resist the heavy impact stresses caused by American locomotives. It would be observed, however, that many highway-bridges constructed of reinforced concrete in Europe had cracked, showing that the sections were too light.

Concrete was a material of great value when properly used, but it lacked tensile strength, and therefore it was often advisable to place rods in plain concrete in order to take care of the tension that would always be found in any structure. It did not follow because reinforced concrete was economical in one part of a structure that it was economical throughout; in fact, the designer should use the materials to their best advantage, either as plain or as reinforced concrete. As an illustration of this, it had been demonstrated that the spirally reinforced column, while of high ultimate strength, was no stronger than a column of a richer mixture when the elastic limit was considered, and that the plain concrete column of a rich mixture with only sufficient rods near its outer edge to take care of eccentric loads was the most economical form of column.

There seemed to be some apprehension, among engineers who must have paid little attention to reinforced concrete, concerning the liability of steel in the concrete to rust should water find its way into the concrete. In the United States it had been thoroughly demonstrated by many tests that when steel was completely embedded in concrete as in beams and slabs, no rust formed on the bars; and bars with a slight film of rust on them placed in concrete which had been alternately immersed in water and dried in the air, had been found to be free of rust when the concrete was broken open after several years. Very thorough tests made in Germany with full-sized beams immersed in weak sulphuric-acid solutions and injurious gases, and subjected to loads which caused the steel in the beams to be stressed to just below the elastic limit, had shown that even under those circumstances, when the underside of the concrete covering the rods was necessarily cracked at short intervals along the entire length of the beams, no damage was done to the steel; whereas when the beams were subjected to loads which caused stresses in the steel exceeding its elastic limit, the steel was found to be rusted after going through the same test. In this test deformed bars of high elastic limit were used, consequently the cracks in the concrete were evenly distributed along the entire underside of the beam; but had plain bars been used, this would not have been the case.

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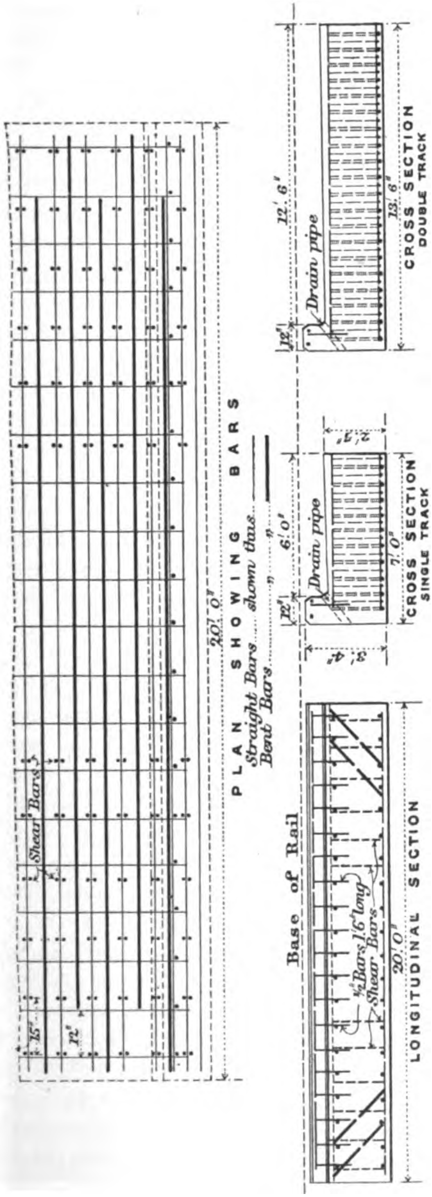
Figs. 15.


Shear Bars
15"
12"
20' 0"
PLAN SHOWING BARS
Straight Bars shown thus
Bent Bars " "
Base of Rail
½" Bars 1' 6" long
Shear Bars
20' 0"
LONGITUDINAL SECTION
3' 4"
12"
6' 0"
Drain pipe
7' 0"
2' 3"
CROSS SECTION
SINGLE TRACK
12"
12' 6"
Drain pipe
13' 6"
CROSS SECTION
DOUBLE TRACK

On American railways it was considered economical to construct all culverts and small bridges with ballasted floors, for the reason

Mr. Cunningham. that open floors composed of wooden sleepers resting on the steel-work were a source of danger and costly to maintain; and it could be shown that a bridge with a ballasted floor, though slightly more expensive in first cost, was far cheaper ultimately than a bridge with an open floor: hence reinforced-concrete slabs carrying ballast and ordinary sleepers were used in preference to steel I beams on which were placed wooden sleepers to carry the rails. The wooden deck of this description cost in the United States $2·50 per lineal foot of track in place, and required renewal at least every 7 years; and when it was necessary to provide new sleepers or new timbers, or to surface or align the track on the bridge it was necessary to send, sometimes from long distances, a highly-paid bridge-carpenter gang to do the work, while on ballasted floors the low-paid track-gang did it.

In order to show the slight differences in design between the slabs shown in the Paper and others designed in accordance with up-to-date specifications, he appended *Figs. 15* (p. 191), which illustrated some of the present standard slabs used on the Wabash Railroad. These slabs were designed in accordance with specifications recommended by the American Railway Engineering and Maintenance of Way Association, and it would be seen that, although they were not quite as thick as those formerly used, more shear-bars were provided. In order that the design of these slabs might be checked, as they would undoubtedly still appear heavy to European engineers, tables of stresses caused by the locomotive-loads were given, together with that part of the recommended specifications relating to design and allowable unit stresses. Particular attention was called to the slight differences between the specifications and unit stresses given in the Paper and those now adopted, and especially to that part referring to diagonal tension. At the time the older specification was used, no practical tests had been made to show the amount of steel necessary to take care of the longitudinal shear; consequently it was assumed that if shear-bars were used, the shear in the concrete could be permitted to run up to 100 lbs. per square inch, and the steel was not taken into consideration; whereas the later specifications gave a more rational way of taking care of such stresses. It was astonishing to those engineers who had used reinforced concrete and knew its merits that there should still be anyone who failed to see its good points; but the number of such engineers was diminishing rapidly, and the time would soon come when it would rarely be found that the structural engineer was unable to design reinforced-concrete structures properly.

[ADDENDUM.

ADDENDUM.

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RECOMMENDED PRACTICE FOR DESIGNING REINFORCED-CONCRETE STRUCTURES AS ADOPTED BY THE AMERICAN RAILWAY-ENGINEERING AND MAINTENANCE-OF-WAY ASSOCIATION, MARCH, 1910.

Material.—The materials and workmanship for reinforced concrete should meet the requirements of the "Specifications for Plain and Reinforced Concrete," presented in this Report of the Committee on Masonry.

The concrete recommended for general use is a mixture of one part of cement to six parts of fine and coarse aggregates. A richer mixture will be found advantageous for special conditions.

Dead Load.—The dead load is to include the estimated weight of the structure and all other fixed loads and forces acting upon the structure.

Live Load.—The live load is to include all variable and moving loads or forces acting upon the structure in any direction.

Impact.—As the working stresses herein recommended are for static loads, the dynamic effect of moving loads is to be added to the live-load stresses.

Span-Lengths.—The span-length for beams and slabs is to be taken as the distance from centre to centre of the supports, but not to exceed the clear span plus the depth of beam or slab.

Internal Stresses.—The internal stresses are to be calculated upon the basis of the following assumptions :—

(*a*) A plane section before bending remains plane after bending.

(*b*) The distribution of compressive stresses in members subject to bending is rectilinear.

(*c*) The ratio of the moduli of elasticity of steel and concrete is 15.

(*d*) The tensile stresses in the concrete are neglected in calculating the moment of resistance of beams.

(*e*) The initial stress in the reinforcement due to contraction or expansion in the concrete is neglected.

(*f*) The depth of a beam is the distance from the compressive face to the centroid of the tension reinforcement.

(*g*) The effective depth of a beam at any section is the distance from the centroid of the compressive stresses to the centroid of the tension reinforcement.

(*h*) The maximum shearing unit stress in beams is the total shear at the section divided by the product of the width of the section and the effective depth at the section considered. The maximum shearing unit stress is to be used in place of the diagonal tension stress in calculations for web stresses.

(*i*) The bond unit stress is equal to the vertical shear divided by the product of the total perimeter of the reinforcement in the tension side of the beam and the effective depth at the section considered.

(*k*) In concrete columns the concrete to a depth of 1½ inch is to be considered as a protective covering, and is not to be included in the effective section.

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When the maximum shearing stresses exceed the value allowed for the concrete alone, web reinforcement must be provided to aid in carrying the diagonal tension stresses. This web reinforcement may consist of bent bars, or inclined or vertical members, attached to or looped about the horizontal reinforcement. Where inclined members are used, the connection to the horizontal reinforcement shall be such as to ensure against slip.

In the calculation of web reinforcement, when the concrete alone is insufficient to take the diagonal tension, the concrete may be counted upon as carrying a part of the shear. The remainder is to be provided for by means of metal reinforcement consisting of bent bars or stirrups, but preferably both. The requisite amount of such reinforcement may be estimated on the assumption that the entire shear on a section, less the amount assumed to be carried by the concrete, is carried by the reinforcement in a length of beam equal to its depth.

The following recommended working stresses, in pounds per square inch of section, are for use in concrete of such quality as to be capable of developing an average compressive strength of at least 2,000 lbs. per square inch, when tested in cylinders 8 inches in diameter and 16 inches long and 28 days old, under laboratory conditions of manufacture and storage, the mixture being of the same consistency as is used in the field :—

	Lbs. per sq. in.
Structural steel in tension	14,000
High-carbon steel in tension	17,000
Steel in compression, fifteen times the compressive stress in the surrounding concrete.	
Concrete in bearing where the surface is at least twice the loaded area	700
Concrete in direct compression, without reinforcement on lengths not exceeding six times the least width .	450
Concrete in direct compression with not less than 1 per cent. nor over 4 per cent. longitudinal reinforcement on lengths not exceeding twelve times the least width	450
Concrete in compression, on extreme fibre in cross bending	750
Concrete in shear, uncombined with tension or compression in the concrete	120
Note.—The limit of shearing stresses in the concrete, even when thoroughly reinforced for shear and diagonal tension, should not exceed	120
Concrete in shear, where the shearing stress is used as the measure of web stress	40
Bond for plain bars	80
Bond for drawn wire	40
Bond for deformed bars, depending upon form . .	100-150

Mr. Cunningham.

Data for Design of Beams and Slabs in Accordance with the Specifications Adopted by the American Railway-Engineering and Maintenance-of-Way Association.

Loads—

Live load is as stated in the Paper, and assumed to be distributed over 10 feet in width of slab.

Dead load, concrete = 150 lbs. per cubic foot. Assumed weight of 18 inches of ballast with sleepers, rails, etc., = 1,800 lbs. per lineal foot on slab 10 feet wide.

$$\text{Impact} = \frac{\mathrm{L}}{\mathrm{L} + \mathrm{D}}\,\mathrm{L}.$$

Formulas—

n denotes $\frac{\mathrm{E}_s}{\mathrm{E}_c} = 15$.

f_s denotes the unit stress in steel = 17,000 lbs. per square inch.

f_c „ „ „ „ concrete = 750 lbs. „ „ „

a „ „ area of steel = $0{\cdot}0088\,bd$.

p „ „ steel ratio $= \frac{1}{2}\left[\frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{f_c n} + 1\right)}\right] = 0{\cdot}0088$.

k „ „ ratio of the distance of the neutral axis from the compressive face to $d = \sqrt{2pn + (pn)^2} - pn = 0{\cdot}398$.

j „ „ arm of resisting couple $= 1 - \frac{1}{3}k = 0{\cdot}867$.

M_s „ „ moment of resistance $= af_s jd = 130\,bd^2$.

M_s for slab 10 feet wide $= 15{,}600d^2$.

M_s for table of moments which is given in lb.-ft. $= 1{,}300d^2$.

d denotes the depth of slab to centre of steel, in inches.

v „ „ maximum horizontal or vertical shearing stress per unit area $= \frac{\mathrm{V}}{0{\cdot}87\,bd}$.

Concrete proportions—

1 cement, 2 sand, and 4 gravel.

Steel rods—

Rods to be deformed and to have an elastic limit of not less than 54,000 lbs. per square inch.

Mr. Cunningham.

Maximum Shear on Slabs 10 Feet Wide.

Clear Span.	Assumed Span Centre to Centre of Bearing.	Shear = V in Lbs.			
		Dead Load.	Live Load.	Impact.	Total.
Feet.	Feet.				
1	2	3,000	50,000	47,200	100,200
2	3	4,700	50,000	45,700	100,400
3	4	6,600	50,000	44,200	100,800
4	5	8,600	50,000	42,700	101,300
5	6	10,700	58,300	47,700	116,700
6	7	13,300	64,300	53,300	130,900
7	8	16,200	68,800	55,700	141,700
8	9	18,800	72,200	57,300	148,300
9	10	21,500	75,000	58,100	154,600
10	11	25,900	81,800	62,100	169,800
11	12	27,300	87,500	66,700	181,500
12	13	31,200	92,300	69,000	192,500
13	14	35,400	96,500	70,600	202,500
14	15	39,800	100,000	71,500	211,300
15	16	44,400	106,300	74,300	225,000
16	17	48,200	111,800	78,100	238,100
17	18	53,300	116,800	80,200	250,300
18	19	57,500	121,000	82,000	260,500
19	20	63,000	125,000	83,100	271,100
20	21	68,800	128,500	83,700	281,100
21	22	74,800	131,800	84,100	290,700
22	23	81,100	134,800	84,200	300,100
23	24	87,600	137,500	84,000	309,100
24	25	94,400	141,300	84,700	320,400
25	26	101,400	144,800	85,200	331,400
26	27	107,000	148,000	85,900	340,900
27	28	112,700	151,000	86,500	350,200
28	29	118,500	154,000	87,000	359,500
29	30	124,500	157,500	88,000	370,000
30	31	130,600	161,000	88,900	380,300

Mr. Cunningham.

MAXIMUM MOMENT ON SLABS 10 FEET WIDE.

Clear Span.	Assumed Span Centre to Centre of Bearing.	Moment in Lb.-Ft. = M.			
		Dead Load.	Live Load.	Impact.	Total.
Feet.	Feet.				
1	2	1,500	25,000	23,600	50,100
2	3	3,500	37,500	34,300	75,300
3	4	6,600	50,000	44,100	100,700
4	5	10,800	62,500	53,200	126,500
5	6	16,100	75,000	61,700	152,800
6	7	23,300	87,500	69,100	179,900
7	8	32,400	100,000	75,500	207,900
8	9	42,300	112,500	81,900	236,700
9	10	53,800	140,700	101,800	297,300
10	11	71,200	164,300	114,600	350,100
11	12	81,900	200,000	141,900	423,800
12	13	101,400	237,500	166,400	505,300
13	14	123,900	275,000	200,600	599,500
14	15	149,300	312,500	209,300	681,100
15	16	177,600	350,000	232,100	759,700
16	17	204,900	387,500	253,500	845,900
17	18	239,900	425,000	271,700	936,600
18	19	273,100	466,300	293,900	1,033,300
19	20	315,000	515,800	320,200	1,151,000
20	21	361,200	565,000	344,700	1,270,900
21	22	411,400	614,300	369,900	1,395,600
22	23	466,300	663,500	389,700	1,519,500
23	24	525,600	713,000	410,500	1,649,100
24	25	590,000	762,500	430,600	1,783,100
25	26	659,100	812,000	441,400	1,912,500
26	27	722,300	861,200	468,400	2,051,900
27	28	788,900	913,800	490,400	2,193,100
28	29	859,100	970,000	513,900	2,343,000
29	30	933,800	1,026,200	532,200	2,492,200
30	31	1,012,200	1,082,200	562,000	2,656,400

Mr. Cunningham.

DEPTH OF SLAB, SHEARING STRESSES, ETC.

Length of Slab.	Vertical Shearing Stress.	Area of Steel required per Foot width of Slab.[1]	Depth of Slab to Centre of Steel.	Total Average Depth of Slab.	Rods used.	
					Diameter.	Spacing.
Feet.	Lbs. per Square Inch.	Square Inches.	Inches.	Inches.	Inches.	Inches.
3	155·0	0·66	6·2	8	⅝	5½
4	126·6	0·81	7·6	10	⅝	4½
5	109·7	0·93	8·8	11	⅝	4
6	98·4	1·05	9·9	12	⅝	3½
7	103·5	1·04	10·8	13	⅝	3½
8	107·2	1·24	11·7	14	¾	4½
9	115·4	1·33	12·6	15	¾	4
10	105·2	1·42	13·5	16	¾	4
11	98·1	1·60	15·1	17	⅞	4½
12	99·2	1·73	16·4	19	⅞	4
13	96·6	1·90	18·0	20	⅞	4
14	93·5	2·08	19·7	22	1	4½
15	90·2	2·27	21·5	24	1	4
16	88·1	2·42	22·9	25	1	4
17	89·1	2·56	24·2	26	1	3½
18	89·1	2·70	25·6	28	1	3½
19	89·8	2·83	26·7	29	1⅛	4
20	88·5	2·98	28·2	30	1⅛	4
21	87·4	3·14	29·7	32	1⅛	3½
22	86·0	3·31	31·3	34	1⅛	3½
23	84·9	3·46	32·8	35	1¼	4
24	84·0	3·61	34·2	37	1¼	4
25	83·2	3·79	35·6	38	1¼	3½
26	82·9	3·91	37·0	40	1¼	3½
27	82·7	4·06	38·4	41	1¼	3½
28	82·4	4·18	39·6	42	1¼	3¼
29	81·6	4·34	41·1	43	1¼	3
30	81·0	4·49	42·5	45	1¼	3
31	80·9	4·63	43·8	47	1¼	3
32	80·6	4·77	45·2	48	1¼	3

NOTE.—The clear span is 2 feet less than the length of the slab in each case.

[1] The area of steel shown in the third column refers to the longitudinal reinforcing bars in the bottom of the slab.

ANNUAL GENERAL MEETING.

26 April, 1910.

JAMES CHARLES INGLIS, President,
in the Chair.

The Notice convening the Meeting was taken as read, as well as the Minutes of the Annual General Meeting of the 27th April, 1909, which the President was authorized to sign.

The Report of the Council upon the Proceedings of The Institution during the Session 1909–1910 was read, the Statement of Accounts being taken as read.

After consideration, it was resolved,—That the Report of the Council be received and approved, and that it be printed in the Minutes of Proceedings.

The Scrutineers reported the election of the Council for 1910–1911 as follows[1]:—

President.

ALEXANDER SIEMENS.

Vice-Presidents.

William Cawthorne Unwin, B.Sc., LL.D., F.R.S.
Robert Elliott-Cooper.
Anthony George Lyster.
Cuthbert Arthur Brereton.

Other Members of Council.

John Audley Frederick Aspinall, M.Eng.
Benjamin Hall Blyth, M.A.
John Alexander Brodie, M.Eng.
William Booth Bryan.
Rookes Evelyn Bell Crompton, C.B.
William Davidson.
Edward Bayzand Ellington.
Maurice Fitzmaurice, C.M.G., M.A., M.A.I., LL.D.
John Purser Griffith.
Sir Robert Abbott Hadfield, F.R.S.
Charles Augustus Harrison, D.Sc.
Walter Hunter.
George Robert Jebb.
Harry Edward Jones.
Sir William Thomas Lewis, *Bart.*, K.C.V.O.
Hugh David Lumsden.
Sir Thomas Matthews.
Hon. Charles Algernon Parsons, C.B., M.A., D.Sc., LL.D., D.Eng., F.R.S.
Alexander Ross.
John Wallis Shores, C.M.G.
Francis Joseph Edward Spring, C.I.E., M.A.I.
John Strain.
Sir Frederick Robert Upcott, K.C.V.O., C.S.I.
Sir Philip Watts, K.C.B., LL.D., F.R.S.
William Barton Worthington, B.Sc.
Alfred Fernandez Yarrow.

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[1] The Council commence their year of office on the first Tuesday in November, 1910.

Resolved,—That the thanks of the Meeting be given to the Scrutineers, and that the Ballot-Papers be destroyed.

Mr. F. Hudleston responded on behalf of the Scrutineers.

Resolved,—That the thanks of The Institution be given to Messrs. P. D. Griffiths and M. F. Wilson, for their care in auditing the Accounts for the past financial year; and that Messrs. P. D. Griffiths and Basil Mott be appointed Auditors for the current financial year.

Resolved,—That the thanks of this Meeting be accorded to Mr. James C. Inglis, President, for his conduct of the business as Chairman of the Meeting.

The President acknowledged the Resolution.

The proceedings then ended.

REPORT OF THE COUNCIL, 1909–1910.

In presenting this Report upon the affairs of The Institution during the year now ended, the Council have to allude to the development of some matters of interest and importance, contained in their Report a year ago.

Professional Conduct.

One of the principal of these matters has been the introduction as part of the By-laws of the Rules of Professional Conduct, which were adopted at the Special General Meeting held on the 22nd February. In formulating these regulations, the Council availed themselves of help afforded by many suggestions and observations which were received from members, both at home and abroad, upon the draft which was circulated last June.

It is not to be supposed that the literal observance of any such code could, alone, entitle a profession to respect and esteem; but the Council believe it to be desirable that a statement of the Rules should be formally incorporated in the Constitution, as in their spirit is reflected the honourable dealing which has conduced so greatly to the prestige of membership of The Institution.

It is, perhaps, appropriate to allude at this stage to the position of the Council in regard to the administration of these rules. Any action by the Council must necessarily be based upon a properly supported allegation of breach of the regulations, or of professional misconduct, brought before them; and the interpretation of the terms and intention of the regulations is to rest with the Council, who would naturally treat any such allegation in a broad and common-sense way. The solicitation of professional work by means of public advertisement was not included specifically in the Regulations, for reasons which were stated to the members in the notice convening the Special Meeting referred to; but the Council do not hesitate to affirm again the repugnance with which members of The Institution generally view advertising of an unprofessional character. The question how far such advertising may form improper solicitation of professional work must be determined on the merits of any individual case.

Conditions of Election.

Another step of vital importance to The Institution has been the amendment of the rules relating to Admission, Election and Transfer. These amendments define somewhat closely what is meant by the regular training of a Civil Engineer, which can be

Elections and Admissions.

properly accepted as qualifying a person for the exercise of that profession. This step may be regarded as complementary to the Examination system established by The Institution, when it laid down definitely its requirements as to general and scientific knowledge as part of the training of engineers, in that it outlines clearly what practical training is also required to fulfil the conditions of Election. It is believed that the new Forms, copies of which have been recently issued to the members, will be found to facilitate the clear presentation of the qualifications of candidates for election and admission.

Notwithstanding the somewhat more stringent conditions thus gradually imposed upon candidates, it is with no little satisfaction that the Council find that the numbers seeking to enter as Corporate Members and Students, have been larger than in any previous year: 487 new applications for Election and 533 applications for Admission of Students have been received in the Session. During that period the Council have actually had under their consideration 571 applications for election.

The attendance at the Examinations has also been well maintained, 500 candidates having sat for the Associate Membership, and 256 for the Studentship Examinations.

The Premises.

The progress of the arrangements for the rebuilding of the premises has been communicated to the members from time to time during the year.[1] Competitive designs were invited last summer from six selected architects, and that of Mr. James Miller, F.R.I.B.A., was chosen, its estimated cost comparing favourably with the forecast given to the members in 1908. Mr. Miller has been accordingly appointed the Architect, and, subject to some modifications of the internal arrangement shown in his plans, which the Council considered desirable, preparations are being made to invite tenders for the erection of the new building at an early date. It is expected that the work will be begun in the coming summer.

As many of the members have not had opportunity of inspecting the designs which were exhibited at The Institution last month, the Council propose to issue shortly a lithographed reproduction of the elevation and main-floor plans, as now settled.

Building Fund.

The Council desire to express their gratification with the substantial support which is being accorded by the members to the special Building Fund; and it may be not inappropriate here to mention again the conditions under which this Fund is being raised. The terms made with the Government for the reinstatement of The Institution, as communicated to the members at the Special Meeting held on the 3rd April, 1908, were fair and reasonable. The contemplated enlargement of the premises, however, and especially the additional feature of a great Hall of a worthy character, is a matter beyond such reinstatement; and the members have been asked, therefore, for some special financial aid to the project, so as to avoid trenching too far upon the capital funds of The Institution. With these views, the Council desire that a sum of at least £15,000 beyond the amount (about £15,000) already subscribed should be forthcoming, and they invite donations accordingly from members and supporters of The Institution.

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[1] A drawing of the front of the present building forms a frontispiece to this volume.

Reinforced Concrete.

Prominent among questions touched upon in the last Annual Report was the investigation of the use of reinforced concrete in engineering construction. The Committee appointed for that purpose has endeavoured in the first instance to analyse the information at present available in regard to this material, and the somewhat complex conditions of its use in this country and abroad. It has now presented to the Council an Interim Report dealing with this stage of the inquiry, with a view to its being placed in the hands of the members at an early date. The results thus arrived at point to the desirability of further action in the direction of experiment to supplement existing data.

Committee on Steam Engine and Boiler Trials.

Acting on a suggestion received from the surviving members of the Committee on Steam-Engine and Boiler Trials, the last Report of which was published in Vol. cl. of the Minutes of Proceedings, that Committee has been reconstituted by the Council to consider a revision of the questions dealt with in the Report.

Building Operations and Engineering Works Bill, 1909.

Among matters which perhaps concern The Institution's immediate objects less than the foregoing, but nevertheless affect the interests of the members in an important degree, are two questions which have engaged the careful attention of the Council.

The first of these was a proposal contained in a Bill entitled The Building Operations and Engineering Works Bill, 1909, which was introduced in the House of Commons last year. The terms of the Bill, and the statements made when it was introduced, appeared to the Council likely to prejudice seriously the position of engineers and of contractors engaged in the construction of engineering works, in respect to control of and responsibility for such operations. Accordingly the Council waited by Deputation upon the Right Hon. Herbert Samuel, M.P., Parliamentary Under Secretary of State for Home Affairs, on the 25th May, 1909, and represented to him the effect of the provisions of the Bill from the engineers' point of view.

Mr. Samuel's reply to the Deputation was of a reassuring character, but the matter was not proceeded with further, as the Bill in question was withdrawn in the House of Commons on the 20th August. A report of the proceedings of the Deputation is in the Institution Library.

The National Federation of Building Trades Employers.

The second question relates to various conditions under which tenders for engineering works are invited, and was submitted to the Council by a Deputation of the National Federation of Building Trades Employers, on the 15th December. A Report of the proceedings at this meeting may also be consulted in the Library. It appeared to the Council that whilst the representations made by the body referred to involved some grievances of a substantial character, the redress of grievances which arose out of conditions of a contract lay between the employer and the contractor rather than between the contractor and the engineer. It appeared further that the Deputation represented mainly contractors engaged with architectural rather than engineering works, or at all events with engineering works of a minor character; and, in the result, the Council felt unable to make a pronouncement upon the several questions submitted to them which would be applicable to engineering works generally.

London Building Acts.

The amendment of the London Building regulations, which was alluded to last year, was effected under conditions that will enable building construction in iron and steel to be undertaken in London on scientific and economical lines. Further, a Tribunal of Appeal has been constituted, upon which Mr. R. Elliott-Cooper, Vice-President, has been appointed by the Council as the representative of this Institution; and satisfactory provision has been made for the formulation of rules as to reinforced concrete when such action is deemed desirable by the London County Council.

Royal Reserve Engineers.

Forty-three appointments have now been made to the Royal Reserve Engineers; and the Council have been much gratified and encouraged by the reports which have reached them about the young officers appointed. They have had further reason for satisfaction in the favourable attitude of the India Office and of other Government Departments to a request that some special consideration might be accorded to such of these officers as may apply for civil engineering posts in those Departments after the completion of their terms of training.

Appointments and Nominations.

During the year the Council have, in response to requests received by them, appointed Mr. James Charles Inglis, the President, to be a representative of The Institution on the Court of the newly established University of Bristol, and have appointed Mr. William

Rich in a similar capacity on the Governing Body of the new School of Metalliferous Mining, Cornwall. They have also reappointed Sir William White to represent The Institution on the Court of Governors of the Sheffield University, and Dr. W. C. Unwin on the governing body of the Imperial College of Science and Technology, in the place of Sir Alexander Kennedy, resigned.

Engineering Institutions.

The attention of the Council has been drawn of late to several proposals for the incorporation of engineering societies, either newly formed or under altered conditions. Whilst the Council are entirely sympathetic with any steps taken to extend the usefulness of associations of engineers, they view with disfavour the multiplication of titles which tend to confusion with those of the leading representative Institutions.

Registration.

The Council have not been unmindful of the observations and suggestions of members as to the advisability of drawing public attention to The Institution's attitude towards questions of professional etiquette, and also as to seeking to secure legal recognition of the status of engineers possessing satisfactory professional qualifications. The Council believe that as regards registration, whilst doubtless some members favour that course, it is not clear that the result of steps in that direction would be advantageous to the members of The Institution or to engineers generally.

Council.

In preparing the Balloting-list, the Council have, as heretofore, paid very careful attention to the considerations of local representation and of personal distinction in the several branches of Civil Engineering, which are accepted as influencing in an important measure the composition of the Institution Council; and, following the usual practice, they have included the names of members resident in the great dependencies of the Empire, being of opinion that such representation accords generally with the wishes of those who practise there, and is in the best interests of The Institution.

THE ROLL.

Enumeration.

The total Roll of The Institution on the 31st March, 1910, stood at 9,057, the changes which took place in it during the financial year ended on that day being shown in the following Table. The elections comprised 38 Members, 279 Associate Members, and 5 Associates; 415 candidates were admitted as Students, and the names of 2 Members, 7 Associate Members and 2 Students were restored to the register. From this addition of 748 must be deducted the deaths, resignations and erasures during the year; the Students elected Associate Members; and those who, having

	1 April, 1908, to 31 March, 1909.						1 April, 1909, to 31 March, 1910.					
——	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.
Numbers at commencement	19	2283	4457	263	1551	8573	20	2279	4592	255	1697	8843
Transferred to Members	..	50	50	..	..		..	67	67	..	..	
Elections	1	15	285	7	..		..	38	279	5	..	
Admissions	..	..	..	..	423	736	..	..	..	..	415	748
Restored to Register	..	2	2	..	1		..	2	7	..	2	
Deceased	..	46	40	10	3		1	47	38	9	5	
Resigned	..	15	35	3	29		..	31	32	7	21	
Erased	..	10	27	2	20		..	5	30	..	20	
Elected Associate Members	..	..	..	..	76		..	..	..	..	88	
Removed—over age	..	..	..	..	148	466	..	..	..	..	194	534
Failed to complete election	..	..	..	..	..		..	..	2	..	..	
Failed to complete admission	..	..	..	..	2	270	..	..	..	..	4	214
Numbers at termination	20	2279	4592	255	1697	8843	19	2303	4709	244	1782	9057

passed the age-limit of that class, ceased to be connected with The Institution; amounting to 534 in all, leaving a net increase of 214.

At the date of this Report, the total Roll of The Institution is 9,136.

Among the deceases, the Council record with especial regret those of H.M. Leopold II, King of the Belgians, Honorary Member, Sir Edward Leader Williams, former Vice-President, Dr. George Frederick Deacon, Member of Council, and Dr. Bindon Blood Stoney, Mr. William John Cudworth and Mr. Charles William Hodson, former Members of Council.

Deaths. The full list of deceases is as follows:—

Hon. Member.—H.M. Leopold II, King of the Belgians.

Members.—Richard Aylmer; John George Barkley; Augustus William Harvey Bellingham; Henry Bramall; George Broadrick; Arthur Cardwell Brown; Mathew Buchan; William Buchanan; Leffert Lefferts Buck; Patrick Forstall Comber; William John Cudworth; George Frederick Deacon; Henry Tanner Ferguson; Walter Faithfull Garland; Alfred Edward Garwood; Malcolm Graham; Arthur Greenwood; Robert Haire; Timothy Harrington; Charles William Hodson, C.S.I.; John Howell Hosgood; Arthur John Hughes, C.I.E.;

John Ellis Hughes; John James Impett; Richard Laybourne; James Chatburn Madeley; William Metcalf; Othniel Foster Nichols; *Sir* Francis Langford O'Callaghan, K.C.M.G., C.S.I., C.I.E.; Harry George Palliser; Edmund Alfred Pontifex; Joseph Quick; Herbert Peter Barrow Rigby; Robert Henry Ryan; Evelyn Francis Sanders; Edmund Du Cane Smithe; Charles Edwin Spooner, C.M.G.; Bindon Blood Stoney; Robert Henry Swindlehurst; George Huntington Thomson; John Deane Tilney, I.S.O.; John Franklyn Tyler; James Walker; George Whale; *Sir* Edward Leader Williams; George Phipps Williams; Thomas Perceval Wilson.

Associate Members.—David Williams Armstrong; Hilary Bauerman; Hugh Meller Bradford; George William Brown; Thomas Common Brown; Henry Goulburn Willoughby Chetwynd; Ratanshaw Dadabhoy; James Deas; Norman Duncan; Henry Bligh Forde; Alfred Maurice Dominic Gallini; Albert William Gertner; Charles James Gunyon; Hugh Erat Harrison; Herbert Richard Hyatt; Arthur Werner Itter; Thomas Bowen Jex-Blake; Charles William Jones; George Ormond Kekewich; Joseph Malcolm Kerr; Richard Lawson Laffère; Percy Lindsay Lascelles; Robert John Dudley MacAllister; Gardiner Henderson Mackillop; Llewelyn Morgan; John Parker (*Hove*); Thomas Mellard Reade; Ben Albert Rhodes; James Sibbald; Stirling Smeaton; William Guthrie Strang; George Taylor; *Rev.* Edmund Henry Tuck; William Andrew McIntosh Valon; Robert Walker; John Henry Wallace; John Webster; James Young.

Associates.—Arthur Stanhope Aldrich; Charles Barclay; John Cresswell; *Sir* Frederick Thorpe Mappin, *Bart.*; James Reid; Horace James Rydon; Charles Smith; Henry Waugh; *Sir* William Lloyd Wise.

The following resignations have been received:— Resignations.

Members.—Charles Toler Burke; George Barker Chalcraft; Henry William Vaughan Colebrook; Jules Ferdinand Conradi; James Young Davidson; Waynman Dixon; John William Dorman; Henry Ewart; Charles Swaine Fahey; Alpin Grant Fowler; Arthur Douglas Fox; Philip Affleck Fraser; Joseph Jones Gardiner; James Dudley Hawks; Thomas Hewson; *Sir* Thomas Higham, K.C.I.E.; Arthur Sampson Jameson; William Benjamin Leggatt; William Henry Le Mesurier; William Cuthbert Lewis; George Henry List; Samuel Joshua Loane; John William Marley; Thomas Bacon Morris; Edmund Penny, C.I.E.; William Shield (*Westminster*); Charles Tickell; Lewis Gordon Tyrrell; William Weaver; Gustav William Wolff, M.P.; James Young.

Associate Members.—Edward Seymour Alexander; Francis William Ashpitel; Frederick Theodore Bagshawe; Frederick Charles Bence-Pembroke; Henry Benest; William Blackshaw; Nicholas Callan; William Lochiel Sapte Lovett Cameron, C.S.I.; Francis Edward Cockshott; Paul Ewens; John William Fletcher; Thomas Robert Gainsford; Arthur Robert Gale; Thomas Harnett Harrisson; William Francis Hassell; Arthur Berriman Hoskings; George Clay Hubback; Edward Fountaine Jacob, C.I.E.; Lawrence Moore Kortwright; Alexander Lade; Charles John Edmonds Large; George Ernest Law; Herbert Augustus Marshall; Alfred Monies; John James Potts; William Eloin Sant; Frank Walter Scott; Arthur Lewis Shackleford; Charles Hockin Treglown; Stephen Watkins (*Wolverhampton*); Rupert Greaves Williams-Ellis; George Shiel Young.

Associates.—Frederick Back; *Sir* Charles Alfred Cripps, K.C.V.O.; Frederick Henry Grinlington, C.M.G.; *Sir* Frederick Harrison; Joseph Henry Judd; Berkeley Paget; *Col. Sir* Colin Campbell Scott-Moncrieff, R.E., *ret.*, K.C.S.I., K.C.M.G.

FINANCE.

Finance. The Statement of Accounts for the year ending the 31st March, 1910, duly audited, is appended to the Report.

The Revenue is as follows:—On General Income Account £29,689 14*s*. 10*d*., of which £24,289 14*s*. 11*d*. represents Subscriptions and Fees apportioned to the Financial Year, £2,576 18*s*. 6*d*. Dividends, Interest, etc., and £1,773 9*s*. Examination Fees.

The expenditure on General Income Account was £26,862 15*s*. 1*d*.—of which the cost of publications was £8,351 2*s*. 6*d*., and the legal and other professional charges, including those of the Institution's petition against the London County Council (General Powers) Bill, 1909, amounted to £1,583 16*s*. 6*d*.

The revenue from the various Trust Funds Investments has been duly collected and applied to the purposes directed under the several Trusts.

The money received on Capital Account, including Admission Fees £3,239 and Life Compositions £89 5*s*., amounted to £3,328 5*s*.

The money received on Special Building Fund Account was £20,830 11*s*. 1*d*., including £4,700 17*s*. 8*d*. contributed by the members and £16,050 received from His Majesty's Commissioners of Works under the Agreements for the acquisition of the New Building site.

The expenditure on Special Building Fund Account amounted to £11,726 17*s*.—including payments made under the Agreements relating to the new site £9,035 18*s*. 4*d*., those for the competitive designs for the New Building £1,260, for architectural assistance in connection with the preliminary schemes £210, and for Surveyors' fees £1,050.

SESSIONAL MEETINGS.

Papers and Discussions. On commencing a second year of office, the President reviewed briefly the recent work of The Institution, the professional status of the Civil Engineer, and the conditions favourable to his activities. This address was delivered to the members at the first Meeting of the Session. The remaining twenty-one Meetings were devoted to the consideration of fifteen original communications. On four occasions two or three Papers were taken simultaneously—a practice that possesses some advantage in the discussion of questions which deserve consideration from a broader point of view than is often suggested by a single communication. The groups of Papers thus dealt with were:—"The Equipment and Working Results of the

Mersey Railway under Steam and under Electric Traction," by J. Shaw, M. Inst. C.E., "The Single-Phase Electrification of the Heysham, Morecambe and Lancaster Branch of the Midland Railway," by J. Dalziel and J. Sayers, and "The Effect of Electrical Operation on the Permanent-Way Maintenance of Railways, as illustrated on the Tynemouth Branches of the North Eastern Railway," by C. A. Harrison, D.Sc., M. Inst. C.E.; "Notes on the Sheffield Water-Supply, and Statistics relating thereto," by L. S. M. Marsh, M. Inst. C.E., and "Statistical and Experimental Data on Filtration," by W. R. Baldwin-Wiseman, Assoc. M. Inst. C.E.; "Birmingham Sewage-Disposal Works," by J. D. Watson, M. Inst. C.E., and "Salisbury Drainage," by W. J. E. Binnie, B.A., M. Inst. C.E.; "The New Clyde Bridge of the Caledonian Railway at Glasgow," by D. A. Matheson, M. Inst. C.E., and "The Queen Alexandra Bridge over the River Wear, Sunderland," by F. C. Buscarlet, Assoc. M. Inst. C.E. and Adam Hunter, M. Inst. C.E. The Council have been gratified to receive again an account of a work of great magnitude and interest, carried out in the United States by a Member of The Institution, namely "The Hudson River Tunnels of the Hudson and Manhattan Railroad Company," by Charles M. Jacobs, M. Inst. C.E. The other Papers discussed during the Session have been:—"Marine Propulsion by Electric Motors," by H. A. Mavor, M. Inst. C.E.; "Railway Signalling in India," by the late Mr. C. W. Hodson, C.S.I., M. Inst. C.E.; "The Design of Rolling Stock for Smooth-Rail Working on Heavy Gradients," by F. W. Bach, M. Inst. C.E.; "The Reconstruction of the Tyne North Pier," by I. C. Barling, M. Inst. C.E.; and "The Use of Reinforced Concrete on the Wabash Railroad, U.S.A.," by E. R. Matthews, Assoc. M. Inst. C.E., and A. O. Cunningham.

The Council have had pleasure in recognizing the special merit of some of these Papers by awards of medals and premiums to Messrs. C. M. Jacobs, D. A. Matheson, J. D. Watson, F. C. Buscarlet, A. Hunter, I. C. Barling, J. Dalziel, J. Sayers, J. Shaw, and the late Mr. C. W. Hodson; and they desire to express their thanks to their colleague, Dr. C. A. Harrison, for the Paper contributed by him. The awards made in respect to Papers published and to be published in Section II of the Proceedings will be announced as usual in the autumn.

The Council have given their best consideration to suggestions that The Institution should permit publication by the Press of its Papers and discussions before they appear in the Proceedings, but in the result they are strongly of opinion that such a departure would be undesirable.

STUDENTS' MEETINGS AND VISITS TO WORKS.

Lectures to Students.

During the Session twelve Students' Meetings have been held in London, four being devoted to the delivery of lectures. At the request of the Council, Mr. H. F. Donaldson, C.B., M. Inst. C.E., delivered two discourses on the subject of "Some Uses of Mechanical Power in Engineering Construction," before the Students in London and at Glasgow, Newcastle and Bristol. The aim of these lectures was to impress upon Students the great importance of detailed investigation of local conditions and adaptation of methods to them, and of organization in the execution of works. The Vernon-Harcourt lectures were delivered this year by Sir R. Hanbury Brown, K.C.M.G., M. Inst. C.E., in London and before the Students' Associations at Birmingham, Manchester and Leeds, the subject being "Irrigation Works." Sir Hanbury Brown's lectures dealt with the principles on which irrigation projects are based and the methods employed in practice in the design and construction of the works required for their realization. These discourses were appreciated highly by those Students and corporate members who heard them, and the Council have to express the special thanks of The Institution to Mr. Donaldson and to Sir Hanbury Brown for their kindness in undertaking the delivery of the lectures.

Meetings and Visits.

The Council regret to observe, however, that, in respect to the attendance at both the Meetings and the Visits to works arranged for the London Students, the improvement recorded last year has not been maintained. At the request of the London Students' Committee, several Saturday visits to works were arranged in addition to those customarily paid on Wednesday afternoons, but the attendance has scarcely been such as to encourage their continuance, especially having regard to the inconvenience necessarily entailed upon those whose establishments are visited.

A special visit has been arranged to the works of the Southampton Docks on Friday, the 29th April, in the evening of which day the Students' Thirty-fifth Annual Dinner will be held in London.

The six Associations of Students in the country, which have been visited as usual by members of the Council in the course of the year, continue to do useful work for the Students in their respective districts.

THE "JAMES FORREST" LECTURE.

James Forrest Lecture.

The eighteenth lecture of this series is to be delivered on the 22nd June by Sir John Gavey—the subject being Recent Developments in Telegraphy and Telephony.

THE LIBRARY.

Contents of Library.

During the year 960 volumes have been added to the Library, of which 597 were presented and 363 were purchased. These figures include 7 volumes of Tracts containing 182 pamphlets, of which 176 were presented and 6 purchased.

A selection of the Drawings bequeathed to The Institution by Telford has been bound in two folio volumes and placed in the Library. A list of the selected Drawings is appended to this Report (p. 237). There is also preserved in the Library a MS. list of all the Drawings bequeathed by Telford, containing a record of the various bodies to which those Drawings that were considered by the Council to be not suitable for preservation by The Institution have been presented.

Through the generosity of Mr. Saxton Noble, M. Inst. C.E., the Library has been presented with the following interesting Drawings and Reports relating to the Thames Tunnel:—

Fifty-five Drawings, many signed by Sir Marc Isambard Brunel.
Chief Engineer's Reports on the Tunnel 1824–44, consisting of MS. letters signed by Sir Marc Isambard Brunel, many of them being in his handwriting.
Resident Engineers' Special Reports, 1835–44 (MS. letters by Richard Beamish and Thomas Page).
Observations by Engineers on Duty, 1836–42 (MSS.).
Thames Tunnel Journal, 23 March—19 November, 1836 (MSS.).
Sinking the Shaft, 16 February—13 October, 1825 (MSS.).

The number of volumes contained in the Library on the 31st March, 1910, was 41,754, made up as follows:—

	Vols.
General Works	15,327
Tracts (in which are bound 15,134 pamphlets)	824
Miscellaneous Reports and Returns	1,027
Parliamentary and State Papers	7,291
Proceedings of Societies	7,480
Journals	9,805
	41,754

Indexes.

The Subject-Index to Vols. cxix to clxx of the Minutes of Proceedings, covering the period 1894–1907, was issued during the summer of 1909. Name-Indexes to Vols. lix to cxviii and Vols. cxix to clxx, respectively, are in an advanced stage of preparation, and it is hoped may be completed before the end of the current year.

The Libraries are open daily from 9.30 to 5.30, but on 53 days during the Session—an average of nearly 3 days weekly—the hour of closing has been extended to 10 p.m., the dates being notified as usual at the commencement of the Session. The Council would be glad to observe fuller advantage taken by the Students of the opportunity for study thus afforded.

[APPENDIXES.

APPEN

BALANCE SHEET

				£	s.	d.
To INSTITUTION CAPITAL ACCOUNT—						
Balance at 31st March, 1910, *as detailed on pages 214 and 215*				205,603	0	7
,, SPECIAL BUILDING FUND—						
Receipts to 31st March, 1910, *as detailed on page 215*	25,446	14	10			
Amounts transferred from Institution Capital Account	17,429	1	8			
				42,875	16	6
,, TRUST FUND CAPITAL ACCOUNTS—						
Telford Fund	7,988	9	4			
Manby Donation	270	0	0			
Miller Fund	4,850	2	4			
Howard Bequest	500	0	0			
Trevithick Memorial	100	0	0			
Crampton Bequest	500	0	0			
"James Forrest" Lecture and Medal Fund	604	14	0			
Palmer Scholarship Fund	1,430	18	0			
John Bayliss Bequest	1,000	0	0			
The Indian Fund	1,148	11	8			
Vernon-Harcourt Bequest	1,000	0	0			
Webb Bequest	1,000	0	0			
Yarrow Educational Fund	10,049	9	6			
				30,442	4	10
,, TRUST FUNDS INCOME ACCOUNTS—						
Balances unexpended—*as detailed on page 218*	483	9	0			
,, ENGINEERING STANDARDS COMMITTEE—						
Balance unexpended, *as detailed on pages 220 and 221*	67	7	2			
				550	16	2
,, INSTITUTION REVENUE IN SUSPENSE—						
Proportion of 1910 Subscriptions applicable to the nine months from 1st April to 31st December, 1910	11,859	18	9			
Subscriptions received in advance . . .	28	10	9			
				11,888	9	6
				£291,360	7	7

DIXES.

31st MARCH, 1910.

							£	s.	d.
By Institution Premises and Buildings, including Buildings now removed, per last Account							124,379	10	0
„ Expenditure on New Buildings to date, *as detailed on pages 214 and 215*							26,556	15	8
„ Institution Investments at cost, *as detailed on page 218*							81,254	12	8
Note.—*The value of these Investments at middle published prices at 31st March, 1910, amounted to* £72,368 16*s.* 4*d.*									
„ Trust Funds Investments, *as detailed on page 219*							30,442	4	10
„ Loans—									
Engineering Standards Committee				500	0	0			
Yarrow Educational Fund				125	0	0			
							625	0	0
„ Cash at Bankers and in Hand—									
Institution Funds—									
Deposit Account	6,100	0	0						
Current Account	5,014	7	9						
In hand	117	19	8						
				11,232	7	5			
Building Fund				16,319	0	10			
Trust Funds — Unexpended Income Balances	483	9	0						
Engineering Standards Committee	67	7	2						
				550	16	2			
							28,102	4	5
							£291,360	7	7

Note.—*No value has been attached, for the purpose of this Statement, to the Books, Furniture, Fittings, Pictures, etc., in the Institution Buildings.*

J. H. T. Tudsbery, *Secretary.*

I have examined the above Balance Sheet and accompanying Accounts with the books and vouchers of the Institution and find the same to be correct and in accordance therewith. I have verified the Securities and Cash Balances, and the Deeds relating to the Premises have been produced to me.

Percival D. Griffiths } Auditor.
*

26th April, 1910.

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* Mr. M. F. Wilson was unable to sign owing to absence from England.

INSTITUTION CAPITAL

EXPENDITURE.

	£	s.	d.
To Amount transferred to Building Fund Account	17,429	1	8
„ Balance, *carried down*	205,603	0	7
	£223,032	2	3

SPECIAL BUILDING FUND

EXPENDITURE.

	£	s.	d.	£	s.	d.
To Expenditure to 1st April, 1909—						
Purchase of 6, Great George Street, Surveyors' Fees, etc.				14,829	18	8
„ Expenditure during the year to 31st March, 1910—						
Under purchase Agreement	9,035	18	4			
Surveyors' Fees	1,050	0	0			
Architects' Fees—Preliminary Schemes	210	0	0			
Competitive Designs for New Buildings	1,260	0	0			
Legal Expenses	71	9	8			
Lithographing Designs and Sundry Expenses	99	9	0			
				11,726	17	0
TOTAL EXPENDITURE TO 31ST MARCH, 1910				26,556	15	8
„ Balance, *carried down*				16,319	0	10
				£42,875	16	6

ACCOUNT, 31st MARCH, 1910.

RECEIPTS.

	£	s.	d.	£	s.	d.
By Balance, 1st April, 1909				216,876	17	6
„ Receipts during the year to 31st March, 1910—						
Admission Fees	3,239	0	0			
Life Compositions	89	5	0			
				3,328	5	0
„ Institution Revenue Account—						
Excess of Income over Expenditure for year to 31st March, 1910				2,826	19	9
				£223,032	2	3
By Balance at 1st April, 1910				205,603	0	7

ACCOUNT, 31st MARCH, 1910.

RECEIPTS.

	£	s.	d.	£	s.	d.
By Contributions received prior to 1st April, 1909	4,616	3	9			
„ Contributions during the year to 31st March, 1910	4,700	17	8			
„ Amount received from the Commissioner of Works towards cost of New Buildings, etc.	16,050	0	0			
„ Interest on Deposit	79	13	5			
Total Receipts to 31st March, 1910				25,446	14	10
„ Amount transferred from Institution Capital Account				17,429	1	8
				£42,875	16	6
By Balance at 1st April, 1910, *represented by Cash at Bank on Building Fund Account*				16,319	0	10

INSTITUTION REVENUE ACCOUNT

EXPENDITURE.

	£	s.	d.	£	s.	d.
To HOUSE AND ESTABLISHMENT CHARGES—						
Repairs and Maintenance of Buildings	78	11	8			
Rates, Taxes and Insurance	1,223	2	5			
Water Charges (including lifts) Telephone Rent, etc.	196	2	0			
Lighting, Warming and Ventilating	486	1	6			
Fixtures and Furniture	45	10	0			
Refreshments at Meetings	89	7	10			
Assistance at Meetings	49	14	0			
Household Expenses	762	15	8			
				2,931	5	1
,, SALARIES AND RETIRING ALLOWANCES—						
Salaries	3,300	0	0			
Clerks, Messengers and Housekeeper	2,126	18	0			
Retiring Allowances and Donations	1,442	10	0			
				6,869	8	0
,, STATIONERY, POSTAGES, ETC.—						
Stationery and Printing	1,649	19	11			
Postages, Telegrams and Parcels	578	17	10			
				2,228	17	9
,, PUBLICATIONS—						
Minutes of Proceedings	8,157	14	6			
Charters, By-laws and List of Members	193	8	0			
				8,351	2	6
,, LIBRARY—						
Books and Periodicals	309	13	5			
Binding	253	18	8			
				563	12	1
,, CONVERSAZIONE, ANNUAL DINNER, ETC.—						
Conversazione	1,419	19	11			
Annual Dinner	267	11	9			
				1,687	11	8
,, STUDENTS' ASSOCIATIONS—						
Grants to Local Associations, Donations to Annual Dinner, Visits, etc.	671	17	11			
Lectures to Local Associations	70	0	0			
				741	17	11
,, EXAMINATION EXPENSES	..			1,033	11	5
,, DIPLOMAS, MEDALS AND PRIZES—						
Diplomas	22	11	0			
Watt Medals	28	0	0			
Stephenson Medals	28	0	0			
Special Prize in connection with Students' Continental Tour, etc.	5	17	0			
				84	8	0
,, EXPENSES OF SPECIAL COMMITTEES—						
Reinforced Concrete	257	8	1			
Steam Engine and Boiler Trials, etc.	1	12	6			
Advisory Committees	17	13	7			
				276	14	2
,, GRANTS, CONTRIBUTIONS, ETC.—						
National Physical Laboratory	500	0	0			
Westminster Hospital	10	10	0			
				510	10	0
,, LEGAL AND OTHER PROFESSIONAL CHARGES—						
Legal and Parliamentary Expenses	1,478	16	6			
Professional Auditor's Fee	105	0	0			
				1,583	16	6
,, BALANCE CARRIED TO CAPITAL ACCOUNT—Excess of Income over Expenditure for year to 31st March, 1910	..			2,826	19	9
				£29,689	14	10

FROM 1st APRIL, 1909, TO 31st MARCH, 1910.

INCOME.

	£	s.	d.	£	s.	d.
By Subscriptions applicable to the Financial Year	..			23,176	14	11
„ Membership Fees	..			1,113	0	0
„ Interest, Dividends, etc.—						
On Institution Investments	2,369	15	1			
On Deposit	55	4	3			
Income Tax Refunded	151	19	2			
				2,576	18	6
„ Examination Fees	..			1,773	9	0
„ Minutes of Proceedings	..			648	16	6
„ Library Fund Donations	..			147	1	0
„ Library Catalogue Sales	..			1	0	0
„ Rents Receivable—6 and 27 Gt. George St.	459	15	0			
Less Outgoings, Rents Payable, Repairs, Rates, Taxes, etc.	207	0	1			
				252	14	11
				£29,689	14	10

LIST OF INSTITUTION INVESTMENTS AT 31st MARCH, 1910.

				COST.		
£	s.	d.		£	s.	d.
10,000	0	0	2½% Consols	9,286	17	1
11,438	8	6	London County 3% Stock	10,760	0	0
4,000	0	0	Manchester Corporation 3% Stock	4,085	1	0
6,000	0	0	Metropolitan 3% Stock	6,517	15	0
3,000	0	0	Metropolitan Water Board "B" 3% Stock . .	2,958	16	0
6,000	0	0	Great Eastern Railway 4% Debenture Stock . .	7,749	18	3
8,000	0	0	Great Northern Railway 3% Debenture Stock. .	7,642	16	4
8,000	0	0	Great Western Railway 4% Debenture Stock . .	10,547	5	0
8,000	0	0	Lancashire and Yorkshire Railway 3% Debenture Stock	7,452	14	8
7,300	0	0	London & North-Western Railway 3% Debenture Stock	6,792	10	5
9,600	0	0	Midland Railway 2½% Debenture Stock . . .	7,460	18	11
			NOTE.—*The value of these Investments at middle published prices at 31st March, 1910, amounted to £72,368 16s. 4d.*	£81,254	12	8

SCHEDULE OF UNEXPENDED INCOME BALANCES ON TRUST FUND ACCOUNTS AT 31st MARCH, 1910.

	£	s.	d.			
Telford Fund, *pages 220–221*	72	19	1			
Manby Donation, *pages 220–221*	9	5	6			
Howard Bequest, *pages 222–223*	34	19	8			
Trevithick Memorial, *pages 222–223*	5	2	6			
Crampton Bequest, *pages 224–225*	13	13	0			
"James Forrest" Lecture and Medal Fund, *pages 224–225*	18	9	9			
Palmer Scholarship Fund, *pages 224–225*	20	18	11			
John Bayliss Bequest, *pages 226–227*	33	7	7			
Indian Fund, *pages 226–227*	26	6	6			
Vernon-Harcourt Bequest, *pages 226–227*	33	0	8			
Webb Bequest, *pages 228–229*	61	9	10			
Yarrow Educational Fund, *pages 228–229*	160	9	4			
				490	2	4
Less—						
Miller Fund Income Balance over expended, *pages 222–223*				6	13	4
				£483	9	0

LIST OF TRUST FUND INVESTMENTS AT 31st MARCH, 1910.

£	s.	d.		£	s.	d.	£	s.	d.
TELFORD FUND.									
5,259	0	3	2½% Consols *at cost*	5,048	9	4			
3,479	12	9	do. Converted from Government Stock *taken at middle published prices at 31st March, 1909*	2,940	0	0	7,988	9	4
MANBY DONATION.									
250	0	0	Great Eastern Railway 4% Redeemable Guaranteed Stock bequeathed *taken at middle published prices at 31st March, 1909*				270	0	0
MILLER FUND.									
5,129	17	5	2½% Consols *at cost*				4,850	2	4
HOWARD BEQUEST.									
551	14	6	2½% Consols *at cost*				500	0	0
TREVITHICK MEMORIAL.									
103	0	0	2½% Consols *at cost*				100	0	0
CRAMPTON BEQUEST.									
512	15	11	2½% Consols *at cost*				500	0	0
"JAMES FORREST" LECTURE AND MEDAL FUND.									
372	0	0	South Eastern Railway 5% Debenture Stock *at cost*				604	14	0
PALMER SCHOLARSHIP.									
1,381	1	6	Metropolitan 3% Stock bequeathed *taken at middle published prices at 31st March, 1909*. . . .	1,298	0	0			
115	4	7	do. purchased with unexpended dividends *at cost*	132	18	0	1,430	18	0
JOHN BAYLISS BEQUEST.									
1,013	17	10	London County 3% Stock *at cost*				1,000	0	0
THE INDIAN FUND. (Formerly comprised in Coopers Hill Trust and the O'Callaghan Medal Fund.)									
1,353	4	2	2½% Consols *at cost*				1,148	11	8
VERNON-HARCOURT BEQUEST.									
1,082	9	10	London County 3% Stock *at cost*				1,000	0	0
WEBB BEQUEST.									
1,055	7	2	Metropolitan Water Board 3% "B" Stock *at cost*				1,000	0	0
YARROW EDUCATIONAL FUND.									
6,728	0	0	Midland Railway 2½% Preference Stock *at cost* .				10,049	9	6
4,220	0	0	North Eastern Railway 4% Preference Stock *at cost*						
							£30,442	4	10

TRUST FUND INCOME

THE ENGINEERING

EXPENDITURE.	£	s.	d.
To Payments to the Committee	3,480	4	6
,, Balance, *carried down*	67	7	2
	£3,547	11	8

TELFORD

EXPENDITURE.	£	s.	d.	£	s.	d.
To Premiums awarded :—						
Session 1907-8	1	3	0			
Session 1908-9	135	15	10			
				136	18	10
,, Medals awarded, Session 1908-9				45	5	0
,, Balance, *carried down*				72	19	1
				£255	2	11

MANBY

EXPENDITURE.	£	s.	d.
To Balance overdrawn, 1st April, 1909		13	3
,, Balance, *carried down*	9	5	6
	£9	18	9

ACCOUNTS (Including Engineering Standards Committee).

STANDARDS COMMITTEE.

INCOME.	£ s. d.	£ s. d.
By Balance, 1st April, 1909		724 11 10
„ Subscriptions	1,238 1 0	
„ Amount received from Committee on Account of Sale of its Publications	1,080 4 6	
„ Interest on Deposit	4 14 4	
		2,322 19 10
„ Loan from Institution's Account		500 0 0
		£3,547 11 8
By Balance, 1st April, 1910		67 7 2

FUND.

INCOME.	£ s. d.	£ s. d.
By Balance, 1st April, 1909		38 1 1
,, Interest on Consols	206 3 6	
,, Income Tax refunded	10 18 4	
		217 1 10
		£255 2 11
By Balance, 1st April, 1910		72 19 1

DONATION.

INCOME.	£ s. d.	£ s. d.
By Interest on Investment	9 8 9	
,, Income Tax refunded	10 0	
		9 18 9
		£9 18 9
By Balance, 1st April, 1910		9 5 6

TRUST FUND

MILLER

EXPENDITURE.

	£	s.	d.	£	s.	d.
To Prizes awarded :—						
Session 1907–8	2	0	0			
Session 1908–9	95	9	11			
				97	9	11
„ Scholarship				40	0	0
				£137	9	11
To Balance Overdrawn, 1st April, 1910				6	13	4

HOWARD

EXPENDITURE.

	£	s.	d.
To Balance, *carried down*	34	19	8
	£34	19	8

TREVITHICK

EXPENDITURE.

	£	s.	d.
To Balance, *carried down*	5	2	6
	£5	2	6

INCOME ACCOUNTS (*contd.*)

FUND.

INCOME.

	£	*s.*	*d.*	£	*s.*	*d.*
By Balance, 1st April, 1909				3	7	10
,, Interest on Consols	121	0	9			
,, Income Tax refunded	6	8	0			
				127	8	9
,, Balance Overdrawn, *carried down*				6	13	4
				£137	9	11

BEQUEST.

INCOME.

	£	*s.*	*d.*	£	*s.*	*d.*
By Balance, 1st April, 1909				21	5	9
,, Interest on Consols	13	0	3			
,, Income Tax refunded		13	8			
				13	13	11
				£34	19	8
By Balance, 1st April, 1910				34	19	8

MEMORIAL.

INCOME.

	£	*s.*	*d.*	£	*s.*	*d.*
By Balance, 1st April, 1909				2	11	1
,, Interest on Consols	2	8	9			
,, Income Tax refunded		2	8			
				2	11	5
				£5	2	6
By Balance, 1st April, 1910				5	2	6

TRUST FUND

CRAMPTON

EXPENDITURE.

	£	s.	d.
To Balance, *carried down*	13	13	0
	£13	13	0

"JAMES FORREST" LECTURE

EXPENDITURE.

	£	s.	d.	£	s.	d.
To Seventeenth Lecture	15	17	0			
,, Medals awarded, Session 1908-9	2	15	0			
				18	12	0
,, Balance, *carried down*				18	9	9
				£37	1	9

PALMER SCHOLAR-

EXPENDITURE.

	£	s.	d.
To Payments to Scholar	44	17	8
,, Balance, *carried down*	20	18	11
	£65	16	7

INCOME ACCOUNTS (*contd.*)

BEQUEST.

INCOME.

	£ s. d.	£ s. d.
By Balance 1st April, 1909		18 2
„ Interest on Consols	12 1 10	
„ Income Tax refunded	13 0	
		12 14 10
		£13 13 0
By Balance, 1st April, 1910		13 13 0

AND MEDAL FUND.

INCOME.

	£ s. d.	£ s. d.
By Balance, 1st April, 1909		18 12 0
„ Interest on Investment	17 11 1	
„ Income Tax refunded	18 8	
		18 9 9
		£37 1 9
By Balance, 1st April, 1910		18 9 9

SHIP FUND.

INCOME.

	£ s. d.	£ s. d.
By Balance 1st April, 1909		21 6 6
„ Interest on Investment	42 5 5	
„ Income Tax refunded	2 4 8	
		44 10 1
		£65 16 7
By Balance, 1st April, 1910		20 18 11

TRUST FUND

JOHN BAYLISS

EXPENDITURE.

	£	s.	d.
To Prizes awarded, Session 1908-9	30	0	0
„ Balance, *carried down*	33	7	7
	£63	7	7

THE INDIAN

(Formerly comprised in the Coopers Hill Trust

EXPENDITURE.

	£	s.	d.
To Premiums awarded, Session 1908-9	33	0	0
„ Balance, *carried down*	26	6	6
	£59	6	6

VERNON-HARCOURT

EXPENDITURE.

	£	s.	d.
To Lectures Session 1908-9	30	0	0
„ Balance, *carried down*	33	0	8
	£63	0	8

INCOME ACCOUNTS (*contd.*)

BEQUEST.

INCOME.

	£	s.	d.	£	s.	d.
By Balance 1st April, 1909				33	4	3
,, Interest on Investment	28	13	0			
,, Income Tax refunded	1	10	4			
				30	3	4
				£63	7	7
By Balance, 1st April, 1910				33	7	7

FUND.

and the O'Callaghan Medal Fund.)

INCOME.

	£	s.	d.	£	s.	d.
By Balance, 1st April, 1909				25	14	2
,, Interest on Consols	31	18	4			
,, Income Tax refunded	1	14	0			
				33	12	4
				£59	6	6
By Balance, 1st April, 1910				26	6	6

BEQUEST.

INCOME.

	£	s.	d.	£	s.	d.
By Balance, 1st April, 1909				30	17	0
,, Interest on Investment	30	11	4			
,, Income Tax refunded	1	12	4			
				32	3	8
				£63	0	8
By Balance, 1st April, 1910				33	0	8

TRUST FUND

WEBB

EXPENDITURE.

	£	s.	d.
To Balance, *carried down*	61	9	10
	£61	9	10

YARROW

EXPENDITURE.

	£	s.	d.
To Scholarships	462	10	0
„ Balance, *carried down*	160	9	4
	£622	19	4

INCOME ACCOUNTS (*contd.*)

BEQUEST.

INCOME.

	£	s.	d.	£	s.	d.
By Balance, 1st April, 1909				30	1	8
,, Interest on Investment	29	16	8			
,, Income Tax refunded	1	11	6			
				31	8	2
				£61	9	10
By Balance, 1st April, 1910				61	9	10

EDUCATIONAL FUND.

INCOME.

	£	s.	d.	£	s.	d.
By Balance, 1st April, 1909				163	1	5
,, Dividends on Investments	318	0	11			
,, Income Tax refunded	16	17	0			
				334	17	11
,, Loan from Institution Account				125	0	0
				£622	19	4
By Balance, 1st April, 1910				160	9	4

MEDALS AND PREMIUMS AWARDED FOR THE SESSION 1908–1909, AND PRESENTED ON THE 2ND NOVEMBER, 1909.

FOR PAPERS READ AND DISCUSSED AT THE ORDINARY MEETINGS.

1. A Telford Gold Medal to Professor Bertram Hopkinson,[1] M.A., B.Sc., M. Inst. C.E., for his Paper "On Heat-Flow and Temperature-Distribution in the Gas-Engine."
2. A Watt Gold Medal to Donald Alexander Matheson, M. Inst. C.E., for his Paper on "Glasgow Central Station Extension."
3. A George Stephenson Gold Medal to Edward Henry Tabor, M. Inst. C.E., for his Paper on "The Rotherhithe Tunnel."
4. A Telford Premium to Stanley John Reed, Assoc. M. Inst. C.E., for his Paper on "The Design of Marine Steam-Turbines."
5. A Telford Premium to Corydon Tyler Purdy, M. Inst. C.E., for his Paper on "The 'New York Times' Building."
6. A Telford Premium to Leslie Augustus Burton Wade, M. Inst. C.E., for his Paper on "Concrete and Masonry Dam Construction in New South Wales."

FOR PAPERS PRINTED IN SECTION II OF THE PROCEEDINGS FOR THE SESSION 1908–1909.

1. A Telford Gold Medal to George Robert Graham Conway, for his Paper on "Aberdeen Main-Drainage Works: Girdleness Outfall Scheme."
2. A George Stephenson Gold Medal to Arthur James Knowles, B.A., M. Inst. C.E., for his Paper on "Road-Bridges over the Nile at Cairo."
3. A Watt Gold Medal to William Charles Popplewell,[2] M.Sc., Assoc. M. Inst. C.E., for his Paper on "Experiments on the Strength Properties of Reinforced-Concrete Beams."
4. The Indian Premium and a Telford Premium to Thomas Richard Nolan, B.E., Assoc. M. Inst. C.E., for his Paper on "The Construction of the Ninth Division (Hills Section) of the Assam-Bengal Railway."

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[1] Has previously received a Watt Medal.

[2] Has previously received a Telford Premium.

5. A Telford Premium to George Hobbs, M. Inst. C.E., for his Paper on "Famagusta Harbour, Cyprus."
6. A Telford Premium to William Cleaver, for his Paper on "Relining of a Portion of Cwm Cerwyn Tunnel on the Port Talbot Railway."
7. A Telford Premium to James Dudley Ward Ball,[1] Assoc. M. Inst. C.E., for his Paper on "Distribution of Locomotive Axle-Loads in relation to Bridges."
8. Telford Premiums to Arnold Hartley Gibson, D.Sc., Assoc. M. Inst. C.E., and Reginald Duncan Gwyther, M.Sc., Stud. Inst. C.E., for their joint Paper on "The Effect of Varying the Degree of Saturation of the Air-Supply to a Suction-Gas Producer."

FOR PAPERS READ BEFORE MEETINGS OF STUDENTS IN LONDON AND AT THE PROVINCIAL ASSOCIATIONS.

1. The James Forrest Medal and a Miller Prize to John Armour Wotherspoon, Stud. Inst. C.E. (Glasgow), for his Paper on "Sewage Disposal Works."
2. The Miller Scholarship, tenable for one year, to James Athersmith Orrell, Stud. Inst. C.E. (Manchester), for his Paper on "The Design of Swing-Bridges."
3. The James Prescott Joule Medal and Miller Prizes to William Ernest Fisher and Edmund Baddeley Wood, B.A., Studs. Inst. C.E. (London), for their joint Paper on "Some Experiments on a Diesel Engine."
4. A Miller Prize to Wilfred Eric Randall Gurney, Stud. Inst. C.E. (London), for his Paper on "Reinforced Concrete on Railways."
5. A Miller Prize to Edwin Guy Lidstone Lovegrove, Stud. Inst. C.E. (London), for his Paper on "Slips in Railway Earthworks."
6. A Miller Prize to John Purser, B.A.I., Stud. Inst. C.E. (Birmingham), for his Paper on "Impact Tests."
7. A Miller Prize to George Chappell Minnitt, Stud. Inst. C.E. (Manchester), for his Paper on "The Design and Construction of Aqueducts."
8. A Miller Prize to Samuel Foster Deacon, Stud. Inst. C.E. (Manchester), for his Paper on "Water-Mains."

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[1] Has previously received a Miller Prize.

LIBRARY OF THE UNIVERSITY OF

9. Miller Prizes to Charles Henry Bradley and Arnold Ernest Marshall, Studs. Inst. C.E. (Birmingham), for their joint Paper on "Influence of Mining Operations upon Engineering Works."

A Special Prize of the value of £5 to Sidney Charles Gladwyn, Stud. Inst. C.E. (Sheffield), for the best Essay on the Works visited during the Students' Continental Tour, 1909.

BAYLISS PRIZE.

Bayliss Prizes, awarded on the results of the October and February Examinations 1908–1909 respectively, to Robert Ferrier McKay, B.Sc., and Harold Vernon Hutt, Studs. Inst. C.E.

ORIGINAL COMMUNICATIONS

RECEIVED BETWEEN 1 APRIL, 1909, AND 31 MARCH, 1910.

AUTHORS.

Airy, W. No. 3,836.—On the Origin of the British Measures of Capacity, Weight and Length.

Baldwin-Wiseman, W. R. No. 3,856.—The Physical Properties of Building Material. With an abstract and 2 tracings.

Balet, J. W. No. 3,880.—Computation of Stresses in Open-Webbed Arches without Hinges. With 4 tracings.

Barling, I. C. No. 3,857.—Tyne North Pier Reconstruction. With an abstract and 2 sheets of drawings.

Barlow, A. No. 3,840. Survey and Soundings of the River Ouse at Aire Mouth. With 1 Map.

Barnhill, J. J. S. No. 3,864.—On the Nature and Amount of a particular Error in Levelling Operations which is induced by the customary Use of Staves of the ordinary pattern. With an abstract and 3 figures in MS.

Bartlett, E. C. No. 3,883.—Investigations relating to the Yield of a Catchment Area in Cape Colony. With 2 tracings.

Belsher, B. J. No. 3,868.—Blyth's Wharf High-Tension Electricity Generating-Station, Stepney. With an abstract, 17 plans and 14 photographs.

Binnie, W. J. E. No. 3,858.—Salisbury Drainage. With an abstract and 11 tracings.

Blackburn, R. H. No. 3,844.—Shoring at Bottoms Mills, Batley. With 2 tracings.

Blackwell, F. O. No. 3,872.—The Schenectady Power Company. With 9 prints and 37 photographs.

Blomfield, H. A. No. 3,852.—The Design of Artificial Harbours on Eastern Coasts of South America, Africa and Australia.

Bruce, A. F. No. 3,884.—Notes on the Labour required for, and Cost of executing some classes of Engineering Work.

Carey, A. E. No. 3,885.—The Winning of Coastal Lands in Holland. With an abstract and 20 tracings.

Corin, W. No. 3,870.—Efficiency Tests of a Hydro-Electric Plant, with general observations upon the Water Power of Tasmania. With 8 tracings and an album of photographs.

AUTHORS.

Cunningham, A. O. See E. R. Matthews.

Dalziel, J., and J. Sayers. No. 3,847. The Single-Phase Electrification of the Heysham, Morecambe and Lancaster Branch of the Midland Railway. With 23 drawings, an appendix and 32 diagrams.

Darby, H. No. 3,869.—Town Planning. With an abstract and 4 diagrams.

Davis, A. C. No. 3,845.—Present-Day Economics of the British Portland-Cement Industry.

——, ——. No. 3,851.—The Portland-Cement Industry of the United States and Canada.

Dawson, W. No. 3,876.—On the Strengthening of New Street Station Roof, Birmingham. With an abstract, 8 drawings and 13 photographs.

———, —. No. 3,877.—On the Reconstruction and Widening of Arpley Bridge, Warrington. With an abstract, 8 drawings and 14 photographs.

De Burgh, E. M. No. 3,846.—Failure of a Reinforced-Concrete Reservoir. With 1 drawing and 5 photographs.

Ford, L. P. No. 3,841.—An Indissoluble Material for Marine Works.

Fowle, F. C. No. 3,867.—Straight Run-Cuts of Cant at the Ends of Circular Curves on Railway Tracks. With an abstract and 4 drawings.

Friend, C. A. No. 3,832.—Note on Underground Water of the Guadaira Watershed, Seville. With 2 tracings and 4 diagrams.

Gibson, A. H., and A. Ryan. No. 3,862.—The Resistance to Rotation of Disks in Water at High Speeds. With an abstract, 9 diagrams and an appendix.

Greaves, R. H. No. 3,879.—The Physical Interpretation of "Hardness" as measured by the Shore Scleroscope. With 2 diagrams.

Griffin, A. E. No. 3,861.—"Taikoo" Dockyard, Hong Kong. With 10 drawings.

Harrison, C. A. No. 3,863.—The Effect of Electrical Operation on the Permanent-Way Maintenance of Railways, as illustrated on the Tynemouth Branches of the North Eastern Railway. With 10 drawings.

Harts, Major W. W. No. 3,855.—Harbour Improvement on the Pacific Coast of the United States. With an abstract and 21 maps.

AUTHORS.

Hay, R. No. 3,854.—The Otago Graving Dock, Port Chalmers, New Zealand. With 3 plans and 3 photographs.

Holmes, J. L. No. 3,850.—Dock and Wharf Equipment. With 1 tracing.

Hummel, F. H. No. 3,889.—Testing Centrifugal Pumps. With 2 diagrams.

Jacobs, C. M. No. 3,859.—The Hudson River Tunnels of the Hudson and Manhattan Railroad Company. With an abstract, a bound set of photographic reductions of drawings, and 11 sheets of tracings.

King, C. W. No. 3,881.—Sand Movements at Newcastle Entrance, New South Wales. With an abstract, 1 tracing, 1 sunprint, and 2 photographs.

Larard, C. E. No. 3,848.—The Behaviour of Ductile Material under Torsional Strain with Restoration of Elasticity at Low Temperatures. With 7 tables, 14 diagrams, and 2 photographs.

Lilly, W. E. No. 3,849.—Eccentrically Loaded Columns. With 11 drawings and an abstract.

Lloyd-Jones, C. W. No. 3,882.—The Design of Fish-Plate Rail-Joints. With diagrams in text.

Matthews, E. R., and A. C. Cunningham. No. 3,834.—The Use of Reinforced Concrete on the Wabash Railroad, U.S.A. With an abstract and 28 blue prints.

————, ——. No. 3,878.—Remodelling of the Main Sewerage System of Bridlington. With an abstract, 9 tracings, and 2 photographs.

Melvill, E. H. V. No. 3,887.—Adjusting the Telescope of a Theodolite. With 1 photograph.

——, ————. No. 3,888.—The Reversible or Double Level. With 1 photograph.

Methven, C. W. No. 3,875.—Durban Harbour, Natal. With an abstract and 17 tracings.

Moore, R. St. George. No. 3,853.—Hankow Waterworks. With 1 map, 8 drawings and 4 photographs.

Newlands, J. E. No. 3,871.—Topographical Surveys on the Canadian Pacific Railway Irrigation Scheme. With 6 tracings and 5 photographs.

Oakden, E. No. 3,833.—Brakes for Cranes. With an abstract and 1 drawing.

Pellereau, H. E. No. 3,839.—Note on the Problem of finding the Dip of a Rope, loaded or unloaded.

AUTHORS.

Pettigrew, W. F. No. 3,838.—Renewing the Roller Paths at No. 3 Gates, Ramsden Dock, Barrow-in-Furness. With 3 drawings.

Popplewell, W. C. No. 3,843.—Experiments on the Strength Properties of Reinforced-Concrete Beams. With 7 diagrams and 5 tracings.

Ryan, A. See Gibson, A. H.

Saner, J. A. No. 3,860.—Reconstruction of the Canal-Boat Lift on the River Weaver at Anderton. With 6 tracings, 4 sun-prints, and an album of photographs.

Sayers, J. See Dalziel, J.

Segrais, P. le J. de. No. 3,874.—Reconstruction of Trianon Bridge, Mauritius, in Reinforced Concrete. With 1 tracing, 3 diagrams and 5 photographs.

Southey, F. R. No. 3,886.—The Beacon Hill Tunnel: Kowloon-Canton Railway. With 4 tracings.

Stotherd, C. E. No. 3,837.—The Water-Supply of Jeypore, Rajputana (No. II). With 3 diagrams and 7 photographs.

Trewby, A. No. 3,842.—Walney Bridge, Barrow-in-Furness. With 8 drawings.

Tuckey, W. R. T. No. 3,866.—Yellow River Bridge, Lanchou-fu, Kansuh, China. With an abstract, 6 tracings, 1 map, and photographs.

Walker, H. No. 3,873.—Sewage Works for Places with a Fluctuating Population.

Watson, J. D. No. 3,865.—Birmingham Sewage-Disposal Works. With 5 tracings, 14 plans, and 2 drawings.

Watts, S. T. No. 3,835.—Some Notes on the Occurrence of Gold Shoots. With 1 sheet of drawings and a box of ore specimens.

TELFORD DRAWINGS.

(*Bequeathed by Thomas Telford, F.R.S., President Inst. C.E.* 1820–35.)

2 Vols. folio.

[With MS. List in 3 Vols.]

Vol. I.

1. **Design of an Iron Bridge.** 1824.
2. **Annan Bridge.** Plan and Elevation of a Bridge proposed to be built over the River at the Town of Annan.
3. **Ballater Bridge.** Design for Proposed Ballater Bridge in Timber.

4.–5. **Chester Bridge.** Elevation and Longitudinal Section of a Stone Bridge of one Arch proposed to be erected over the River Dee at Chester.

6. **Clifton Bridge.** Elevation.
7. **Dornoch Firth Bridge.** Plan and Elevation of a Cast-Iron Bridge for the Dornoch Firth. 1810.
8. **Edinburgh.** Design for Dean Bridge of Four Arches. 1830.
9. **Fochabers Bridge.** Plan and Elevation of a Bridge intended to be erected over the River Spey . . . at Fochabers, with a Map of the Site and adjacent grounds showing the space covered by the great Flood of August 1829. 1830.
10. **Menai Bridge.** Design for a Bridge proposed to be erected over the Menai upon the Swilley Rocks.

11.–12. **Morpeth Bridge.** Design for a Bridge over the River Wansbeck at Morpeth, with Transverse Section of the bed of the River showing the Strata and the Piers and Abutments.

13. **Padua.** Elevation of an Iron-Wire Bridge over the Bacchiglione.

14 to 23. **Runcorn.** Design for a Cast-Iron Bridge at Runcorn, and Design, Plans, Elevations and Sections for a Bridge over the River Mersey at Runcorn. 1814, 1817.

24. **Sark River Bridge.** Plan of a Bridge for the River Sark.
25. **Stirling Bridge.** Elevation and Plan of a Bridge proposed to be erected over the River Forth at the Town of Stirling. 1822.
26. **Stonebyres Fall Bridge.** Plan, Elevation and Section of a Design for a Bridge over the River Clyde near the Great Stonebyres Fall.

27 to 29. **TEWKESBURY SEVERN BRIDGE.** Plans, Elevations, Sections and Details. 1824.

30. **YNYS-Y-MOCH BRIDGE.** Designs for a Bridge proposed to be erected over the Menai at Ynys-y-Moch.

VOL. II.

1 to 4. **ENGLISH AND BRISTOL CHANNELS SHIP CANAL, PROPOSED.** Sketch of Entrance. Plans and Section. 1824.

5. **CANAL LOCK.** Plan and Longitudinal Section of the Design for a Canal Lock.
6. **DUMFRIES CANAL LOCK.** Plan and Section.
7. **JUNCTION DOCK LOCK-GATES.** Plan, Section and Elevation.
8. **QUEEN'S DOCK, LIVERPOOL.** Plan of the Cill of the Queen's Dock Gates at Liverpool.

9. **HOLYHEAD HARBOUR.** Plan and Sections of a Graving Dock for Holyhead Harbour. 1823.

10. **EAST AND WEST INDIA DOCKS RAILROAD.** Proposed Railroad from the East and West India Docks into London.
11. **LIVERPOOL AND MANCHESTER RAILWAY.** Section.

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12. 13. Designs for a Monument to be erected to the Memory of Thomas Telford, Esq.

EXTRA MEETING.

22 June, 1910.

JAMES CHARLES INGLIS, President,
in the Chair.

THE "JAMES FORREST" LECTURE, 1910.

The President said he had much pleasure in introducing Sir John Gavey, who was to deliver the eighteenth "James Forrest" Lecture. These Lectures had become quite an institution; the members had listened to a series of interesting orations from various experts who had come before them, and he had no doubt that those who had had the experience of giving a "James Forrest" Lecture found it was not quite so simple a task as it appeared. He was sure they would derive much pleasure that evening from hearing Sir John Gavey deal with the interesting subjects of Telegraphy and Telephony.

The following Lecture was then delivered:—

"Recent Developments in Telegraphy and Telephony."

By Sir John Gavey, C.B., M. Inst. C.E.

The subject selected by the President and Council for to-night's lecture covers such wide ground that to deal with it comprehensively would require a whole course of lectures rather than a single address. I hope, however, to be able to condense my subject-matter so as to give a general view of the most important and promising developments in each branch without exceeding the limits of your patience.

We may, I think, select the ordinary Morse circuit as our starting-point, as by far the largest proportion of the world's telegraphic traffic is still conducted on this system, and, as will be seen later, many of the developments in telegraphic apparatus have been based on the Morse method of signalling. The Morse alphabet and its variants are so well known that it is not necessary to describe them in detail here. It is also unnecessary to dwell on the simple form of Morse circuit, which consists of a battery with one pole to earth, a key for making and breaking the contact between the battery and the line, and at the far end of the latter an electro-magnet which records all the movements of the key, further than to point out that the speed of working, even under the best conditions, is limited by the skill of the operator, and that it rarely exceeds thirty words a minute, the average being lower. For many years this simple form of apparatus or its equivalents served the requirements of most countries, but as the telegraph service grew and the traffic rendered it imperative to erect long lines directly connecting distant cities, the problem of obtaining a greater revenue from the large capital expenditure involved became pressing, and progress was made broadly on three distinct lines of development. In the first, means were designed for the transmission of several messages simultaneously over the same conductor; in the second, by the use of suitable mechanical and electrical devices the actual speed of transmission was raised in overhead wires to ten or twelve times that possible by manual operation, and, finally, type printing and writing systems were devised, which worked with varying degrees of success.

The duplex system admits of the simultaneous transmission of two messages in opposite directions over the same wire. At each end of the circuit is a differential receiver comprising two equally-balanced electro-magnetic coils, one of which is connected at its further end with the line wire and the other with an artificial line which, in its electrical resistance and capacity, is as nearly as possible the equivalent of the actual line. The two inner ends of the magnet coils are joined together, so that when a current is sent from the point of junction it divides equally between the actual and the artificial lines, and the coils being so wound that under these conditions the magnetic effect of one is neutralized by the magnetic effect of the other, the signals originating at either end pass through the home apparatus without affecting it. At the far end, when one station only is signalling, the current passes through one coil alone of the receiving apparatus and the signal is recorded. When, however, both keys are depressed simultaneously, if the batteries are so connected that they aid each other (they may either oppose or aid), then the current which circulates through the line wire and through both of the magnet coils attached thereto is doubled, but as the current which circulates through the magnet coils at each station connected with the respective artificial lines is unaltered, a signal is recorded at each station because the balance at either end is destroyed by the action of the other operator.

The transition from duplex to quadruplex working, viz., the sending of four messages simultaneously over one wire involved but a step. There are two kinds of electro-magnets in use telegraphically, one provided with a simple soft iron armature which is equally attracted by positive or negative currents, and one provided with an armature which is itself a magnet, and which remains unaffected, or rather is repelled against its limiting stop, by a current that induces in the magnet coils a similar polarity to its own, but is attracted by a reverse current which induces an opposite polarity. Either type of magnet may be used for duplex, but both types are used for quadruplex working. The non-polarized magnet responds to a current of considerable strength in either direction, the polarized magnet to a weaker current in one direction only. In practice a single-current key is connected with a powerful battery which is intended to work the non-polarized magnet, but the connections are led through a reversing key, which latter is designed to actuate the polarized magnet. When the single-current key is depressed, the powerful current that circulates through the apparatus actuates the non-polarized relay at the far end, leaving the polarized relay unmoved. When the reversing key alone is depressed, a weak current circulates which actuates the polarized magnet but is too weak to act on the non-polarized instrument. If both are depressed simultaneously the powerful current from the single-current key is reversed by the depression of the double-current key and both instruments at the far end are affected. The receiving apparatus at each end is arranged differentially as in the duplex system, so that the outgoing currents have no effect on either of the receivers at the home end, except when the instruments at both extremities are being worked simultaneously. This arrangement admits of four operators sending four messages simultaneously over one wire. Time does not admit of pursuing this interesting subject further, but it may be stated that as the artificial lines must be as closely as possible the electrical equivalents in resistance and electrostatic capacity of the actual lines, means have to be provided for varying these factors, as the actual lines themselves vary under the influence of weather changes.

Another method, based on entirely different principles, which in theory admits of sending as many as twelve simultaneous messages in one direction, or double that number if duplexed, depends on the superposition of musical vibrations on a telegraphic circuit instead of using steady currents. To effect this result a number of electrically-driven tuning forks, arranged to vibrate at different frequencies, are connected through telegraphic keys with a line wire, so that on depressing any one key a series of electrical vibrations, of the frequency of its companion tuning fork, are sent through the line. At the far end the receivers are of a type that will respond to musical vibrations only, and each receiver is constructed or adjusted to respond to the vibrations of one of the distant tuning forks alone and to no others. If any one key is depressed a simple musical oscillation traverses the line and the receiver in tune responds. But if two or more keys are depressed simultaneously a series of compound curves is transmitted and those receivers that are in tune with the various components of the curves respond and all the others remain unaffected. This system originated in America, but it has been developed and improved by Mercadier in France, where it is said to have given good results recently. In the modern apparatus the receivers consist of so-called mono-telephones, each of which is so made and adjusted as to respond to only one frequency.

A method of multiplex telegraphy based on entirely different principles is the following. The working of the Mercadier system just described has shown that the electrical conditions of an overhead line admit of a far more rapid transmission of readable signals than is possible by ordinary manual operation, and it has also shown that the continuous currents used in general for signalling dots and dashes may be replaced by intermittent currents which, by suitable means, can be translated at the receiving instruments into the ordinary Morse dots and dashes. This line of reasoning explains the following development. A large number of insulated metallic segments are arranged in a circle at each end of the line and are divided into groups, say, of eight each. No. 1 of the first group is metallically connected with No. 1 of all the other groups, and, through the transmitting key, with No. 1 instrument, No. 2 with No. 2 series of contacts and instrument, and so on. Centrally-pivoted arms, provided with metallic brushes which ride over the segments, are joined one to each extremity of the line wire, and it is obvious that if both these arms, starting simultaneously from No. 1, revolve at precisely the same speed, then each segment with its instrument at one end is for a brief period placed in connection with its corresponding segment with its instrument at the distant end. If the sending key of No. 1 instrument is depressed, a brief current is transmitted to its corresponding instrument when the revolving arm reaches No. 1 contact of any group, to be repeated when the arm reaches No. 1 of the next group, and so on throughout the series. Instead, therefore, of a dot being represented by a single current of continuous duration it is replaced by three or four shorter impulses spread over the same period of time, and a dash by three times the number of impulses. By this means it is possible, under favourable conditions, to transmit eight messages simultaneously over one wire. This system, invented by Delaney, was developed to some extent in this country, but owing to various causes it has recently been abandoned. It is, however, referred to here as the principle of revolving commutators is in use in other forms of apparatus to be dealt with later.

The second method of increasing the output of telegraphic wires is the automatic or machine transmitting-instrument, which is typified by the Wheatstone apparatus adopted and perfected by the Post Office in Great Britain. In all instruments of this character a long paper ribbon is perforated by a suitable machine in an arbitrary manner, and the transmitting and receiving apparatus is so designed as to transcribe these perforations, at the distant end, into Morse signals, into similar perforations, into type-printed messages, or even into written characters. In the perforated tape, or slip, as it is termed, used in the Wheatstone system, there are three rows of perforations. The centre row is continuous, as it is used for drawing the slip through the transmitter at the speed at which the latter is running; the upper and lower holes cause the transmitting key to send electrical signals, dots when these rows are vertical, dashes when diagonal. The actual apparatus which effects this object is comparatively simple. Two small vertical plungers, to which an up-and-down motion is imparted by the clockwork of the apparatus, are pivoted to two bent levers connected by horizontal rods with a small vertical transmitting-key, which in one position sends a positive, and in the other a negative current to line. When no slip is in the machine, the plungers as they rise and fall move the key to and fro and send reversals which are received as dots at the far end. When the slip is inserted, if the plungers meet a series of vertical perforations they pass through at each movement and dots are still transmitted, but if they meet diagonal perforations, the movements of the plungers are arrested by the unperforated portion of the paper slip, and the transmitting key is not drawn back until one of the plungers enters the lower diagonal hole of the slip and withdraws the key from its sending position, thus recording a dash.

This Wheatstone system has been very fully developed in the United Kingdom. It is capable of dealing with traffic at a maximum rate of 450 words per minute and it is invaluable for the transmission of news. Thus, in the central office in London, items of news may have to be transmitted to fifty or more towns simultaneously.

Circuits are made up for news transmission, each providing for a number of towns, some of the circuits being of a permanent character and some formed temporarily to meet special requirements. As many as eight Wheatstone slips can be punched simultaneously in one operation and each length of slip is run through the necessary transmitters at the highest speed considered judicious. When long press messages are received, they are divided into sections and each section handed to a separate telegraphist for perforating, so that the transmitting apparatus can be kept to its maximum capacity. Without this useful and adaptable apparatus, it would be almost impossible to deal satisfactorily with the vast amount of news traffic which is sent daily to every town in the kingdom.

For ordinary public-message traffic, on lines of moderate length, where each individual message is short, the Wheatstone system has certain disadvantages, namely, the initial delay in perforating the slip, its distribution, and finally the re-distribution of the received slip amongst the writing telegraphists; for it is obvious that at the high speed at which the Wheatstone machine is worked, several operators are required at each end of the line to keep pace with the apparatus. In practice in this country, for circuits of moderate length, it is generally considered preferable to provide direct Morse apparatus worked simplex, duplex or quadruplex, as circumstances may dictate.

With overhead lines the limit of speed in automatic working is that imposed by the capacity of the receiving apparatus, which, owing to its self-induction (a factor referred to later), obstructs the reception of Morse signals at a higher speed than that named. This difficulty has been overcome by substituting a chemical for an electro-magnet receiver. In this form the current at the received end passes through a long paper ribbon saturated with a solution which is decomposed by a positive current. The Morse signals appear in blue lines on the received slip. It is said that with this method a maximum speed of 1,000 to 1,200 words per minute is possible under favourable conditions, but the difficulty in working at such high speeds, where characters are received in Morse code and have to be transcribed manually, is the division and distribution of the slips amongst the large number of writers necessary to keep abreast of the work, the precautions needed to avoid loss of messages, the injurious effect of brief contacts caused by workmen which result in the loss of several words, and last but not least. the difficulty and delay in obtaining repetitions where errors, false signals or missing words render this necessary.

All the foregoing methods increase the carrying capacity of the wires; in other words, they reduce the capital expenditure per message, but none of them increase the output per operator, nor do they diminish the working cost in the instrument-room; in fact, with high-speed automatic transmission this cost may be higher than with other methods described. The messages have to be prepared by the perforation of the punched slip, telegraphists have to control the sending and receiving apparatus, and the Morse slips, as they are reeled off the receiving apparatus, have to be divided and distributed amongst a number of operators for transcription. The initial preparation of the transmitting slip will always, of course, be necessary in all automatic systems, but inventors have turned their attention to increasing the speed and reducing the cost of transcription at the received end, in the case of manual as well as automatic sending, by the substitution of typing apparatus worked mechanically or electrically for the manual transcription. A very considerable number of instruments has been designed to achieve this end, but one of the earliest which has met with permanent success, and by means of which a very large proportion of the work in Europe and nearly the whole of the transcontinental work is dealt with, is the well-known Hughes type-printing instrument. Each instrument comprises a type-wheel, which consists of a disk on the periphery of which are embossed the letters of the alphabet, numbers and other signals that have to be transmitted; a piano-like key-board on the keys of which are engraved the letters and signals which appear on the type-wheel; and a horizontal plate with a series of slots arranged in a circle. Enclosed in each of these slots and connected with one of the sending-keys is a movable plunger which is raised when the key is depressed, but which normally lies flush with the surface of the plate. An arm carried on a vertical shaft in the centre of the plate is caused to revolve above it by the motive power which drives the instrument, and when a key is depressed this arm is arrested momentarily by the protruding plunger and a current is sent to the distant station. The printing disk and the revolving-arms at both sending- and receiving-stations move in unison, this being effected mainly by the action of a governor, any slight want of coincidence being corrected mechanically each time a letter is printed. Below the type-wheels are long paper ribbons, and it will be understood that, both instruments starting from zero and running uniformly, when a key is depressed and the plunger corresponding with the letter to be signalled is raised, the electrical current is sent to line at the moment that the letter to be printed is immediately over the paper, and if the latter is raised momentarily that letter is printed.

The Hughes method of transmission has many advantages. It provides a clearly-typed message for delivery instead of a written one, it removes a possible source of error in transcription, and it increases the speed of working as compared with the Morse system by about 25 per cent. It can be duplexed and it is used by the Post Office on all its continental wires. It has, however, the disadvantage that a considerable interval of time elapses between the transmission of two consecutive signals owing to the revolving arm having to traverse all the intervening letters. Baudot has obviated this waste of time by adopting the multiple system of telegraphy. He entirely abandons the Hughes method of transmission, and forms an arbitrary signal code with which, by means of five consecutive currents, some plus some minus, in combination, he represents each letter of the alphabet, figures or other signals. By this method four or six channels can be provided simultaneously on one wire, each being worked manually. In the quadruple system, that most generally in vogue, four separate keyboards are used, each containing five keys. Either key when in its normal condition is in a position to send a negative current and when depressed a positive current to line. Each of the five keys of one keyboard is connected with five separate and contiguous metallic contacts arranged in a circle. An inner continuous circle is connected with the line wire. A revolving arm, moving at a uniform speed, connects in regular sequence the individual key sections with the continuous line circle, and during each revolution it transmits through the line the groups of signals formed on each keyboard. At the receiving end a corresponding arrangement, the arm moving at the same speed, connects the line in regular sequence with the four receiving instruments, each set being provided with five polarised relays working five electro-magnets. During each revolution of the arms at the sending and receiving ends, the electro-magnets repeat the positions of the transmitting keys, the armatures of those receiving positive currents being raised and those receiving negative currents depressed, and it remains only to translate these arbitrary code-signals into printed letters. This is effected in the following very ingenious manner. At each receiving instrument two disks, firmly fastened together, have on their peripheries a series of serrations, corresponding with the arbitrary code, which, in certain defined positions, represents the various letters of the alphabet. Riding on the periphery of one of them are five levers termed selectors, each of these levers being so pivoted as to be susceptible of a slight to-and-fro movement on its axle, and also of a transverse movement in the line of the axle. When all the electro-magnets have their armatures raised, the selectors rest on the inner disk, but if a certain number of the armatures are depressed, the corresponding selectors are shifted through the action of intermediate levers to the second disk. As the two disks revolve together the combined selectors at some point drop simultaneously and for a brief interval into the series of serrations which correspond with the letter code signalled from the far end, and this movement and the rapid jerk out of the serrations as the disks revolve causes the letter to be printed from a type-wheel which revolves at the same speed as the disks. The uniform revolution of the contact arms, the coded disks and the type-wheels, are maintained by a clever adaptation of governors and brakes partly electrical, partly mechanical, and provision is likewise made for the lag of the current in transmission over the line.

The Baudot system admits of the transmission of a much larger number of messages over each wire than the Hughes system. It is also more flexible, inasmuch as the various channels it provides can be divided amongst an equal number of towns; thus Paris can use two channels to Lyons and two to Marseilles over a Paris-Lyons circuit extended from Marseilles, and so on. It is largely used in France and has been introduced into this country.

It will have been observed from the foregoing description of typical forms of apparatus that there are three distinct methods of telegraphic transmission with which we are mainly concerned to-night, although others might be mentioned. In the first an arbitrary code of signals is repeated in similar arbitrary signals by which the alphabet is artificially represented and the message is read by a skilled operator; in the second, what may be termed the dial type of apparatus is used, where two type-wheels, moved either mechanically or electrically, revolve isochronously, and they may either show fleeting letters or print them in permanent characters; and in the third an arbitrary set of electrical signals is devised which actuates specially designed apparatus which may reproduce the message in legible characters, printed or even written. The third method has been utilized by various inventors and applied to automatic transmission so as to dispense with manual transcription at the receiving station.

In the latter case a paper ribbon or slip is perforated by punches generally actuated by a specially designed typewriter keyboard, in which the depression of any key causes a series of perforations representing the arbitrary combination of the corresponding letter to appear on the slip. This is passed through an automatic transmitter, the electrical currents corresponding with the perforations are transmitted over the circuit and the distant apparatus is actuated.

Murray has devised a system which has undergone lengthy trials both at home and abroad. His slip has one row of perforations which gears into the moving mechanism of the transmitter, and below this a second series of perforations representing his artificial signalling code, which is of the Baudot type. The transmitting and receiving apparatus must run isochronously, and this is provided for by an original method which time does not admit of explaining in detail. Briefly, however, when the vertical plunger of the transmitter passes through a perforation a positive current is sent over the circuit, and when the plunger is stopped by the paper a negative current circulates. The result at the receiving apparatus is that an exact counterpart of the transmitted slip with its perforations is reproduced, and this perforated slip is passed through and actuates an automatic typewriter which prints the message. The typing machine is fitted with five horizontal plungers which are prolongations of an equal number of slotted combs to which in action a to-and-fro motion is imparted. The perforated slip acts on the principle of the Jacquard loom, and when at a given moment a series of perforations appear, the corresponding plungers move forward and the remainder are arrested by the unperforated part of the paper. The slotted combs are at the moment so grouped that the one letter indicated by the code is actuated by the controlling motive power and it is accordingly printed, and the combs restored to their normal position ready for the next letter. The message is typed in page form. An improved printer, recently devised by Mr. Murray, is shown at work and can be inspected later.

Creed has worked in the same direction, but he uses the ordinary Wheatstone alphabet already described, and of course the Wheatstone transmitter. At the receiving end a perforated Wheatstone slip is reproduced by a punching machine, which, controlled by the reverse currents from the transmitter, and, using compressed air as a motive-power, perforates the received slip at considerable speed. This slip is then passed through an automatic typewriter adapted to work with the Wheatstone alphabet, which types the message on a long slip, to be gummed on the telegraph form. In both these cases the received slip can be inserted in a second automatic transmitter and the message sent on to another town—an advantage in the transmission of news, which frequently has to be redistributed from large provincial centres to other towns having no direct communication with London. Both these systems are in use in the British Post Office. Messrs. Siemens and Halske in Berlin have devised an automatic system in which, by means of suitable apparatus, the message is printed at the receiving end direct by photographic methods. Time precludes even a reference to the ingenious details of the system beyond stating that the final typed characters are produced photographically by an electric spark, which momentarily illuminates the required letter. The alphabet is stencilled on a revolving disk, and the arrangements are such that the electric spark passes only when the letter corresponding with the arbitrary signal is in the right position for printing.

The Western Union Telegraph Company in America, the land of long-distance circuits, has introduced extensively a system known as the Barclay automatic type-printing apparatus. At the sending end the code of electrical signals adopted consists of long and short intervals, either marking or spacing, representing the various letters of the alphabet. Every letter is represented by six current alternations, but the receiving apparatus is so designed that although every current acts on an escape wheel that makes a momentary contact with certain selecting relays, only those of long duration effect any movement in the latter. There are thirty-two printing magnets which act as the keys of a typewriter and cause the message to be printed direct on the receiving instrument.

It may be added that, although any one of the systems described or referred to can be made to do excellent work, the actual selection of the apparatus best calculated to meet the requirements of any country depends entirely on local conditions, length of the lines, character of the work to be dealt with, idiosyncrasies of the staff, etc., and that it would be eminently unwise to dogmatize on the subject. To what extent uniformity of practice is likely to arise in the future is a doubtful question, as each country is likely to progress on the lines which have been firmly established as best adapted to its wants. In this respect telegraphy may be said to occupy a somewhat unique position from the inventor's point of view, there being no tendency, at present to adopt one particular type of apparatus as being best adapted for all purposes.

Writing telegraphs, based on the fact that two ordinates at right angles to one another can be made to describe any curve, have been designed. The telewriter, in which the pen is connected with two arms which follow the movements of the writer, and which in doing so pass over varying resistances and transmit to line currents of varying strength, is well known. At the receiving end two pivoted electro-magnets, placed in a very powerful magnetic field, are deflected over arcs dependent on the strength of the current circulating at any moment. Two arms at right angles to one another are connected with the transcribing pen, one arm being pivoted to each magnet, and the writer's movements are reproduced.

There is only time to refer briefly to the beautiful writing apparatus designed by Pollak and Virag. In this a slip is perforated by suitable means with nine rows of holes of varying sizes, suitable flexible brushes make contact through these holes between batteries and the line wire, and thus cause currents of different electromotive forces and duration to circulate over the line, and to act on two telephone receivers at right angles to one another. Rays of light are reflected from one to the other and on to a photographic slip, and the written messages, which can be transmitted at 600 to 1,000 words a minute, appear developed and fixed on the sensitized paper which emerges from the dark closet of the apparatus. There are numerous type-printing instruments for the distribution of news, which, however, are so well known that further reference to them is unnecessary.

So far we have only considered the apparatus for the transmission and receipt of telegrams on land lines, but before we proceed with our subject it is necessary to deal with the behaviour of the conductors by means of which electrical energy is transmitted from one station to another. In considering this branch of our subject, there are several factors to take into account, each of which may profoundly modify the results according to the circumstances under which the transmission takes place. These factors are conductor-resistance, electrostatic capacity, self-induction and insulator-resistance or its reciprocal leakance.

Resistance wastes the effective electrical energy by degrading it into heat, and its sole effect is to reduce the amount of current available at the distant end. Electrostatic capacity converts a portion of the kinetic energy into potential energy, which is restored to the circuit on the cessation of the current, and it has the effect of retarding the receipt of the first portion of each signal and extending its duration when the line is disconnected from the source of energy. To illustrate this, assume the case of a conductor with high capacity, say a length of submarine cable. If the far end is disconnected, then on making contact with one pole of a battery, the other pole of which is to earth, there is a rush of current into the cable which continues until the latter is charged uniformly to the potential of the battery. When the battery is disconnected from the line and replaced by the earth, the charge leaks out in the form of a reverse current. If, when the battery contact is made, the far end of the line is to earth the initial portion of the current is taken up in establishing the cable charge, whilst only a fraction of its ultimate value trickles out at the distant end a short interval after the battery contact is made. The duration of this interval depends on the capacity and resistance of the line. When the full charge has accumulated along the circuit, the full current dependent on the potential and the resistance flows along the conductor. On replacing the battery by earth the charge flows out at both extremities. Self-induction may be taken as the analogue of mechanical inertia. The first effect when a source of electrical energy is connected with a conductor is to establish a magnetic field in planes at right angles to the conductor, and this retards the period when the full strength of the current is established. Conversely, on the cessation of the primary current, the magnetic field closes in on the conductor and generates a transient current in a direction which tends to prolong the original current. Leakance causes a loss of current which should otherwise be utilized in actuating the distant apparatus. Per contra, it exercises a beneficial effect in high-capacity lines in facilitating the static discharge, but so far it has not been found practicable to utilize the effect without introducing other evils.

These somewhat elementary facts, which of course are well understood, have been referred to so as to maintain the logical sequence of our subject, for they govern the limiting distances and speeds at which electrical communication of all types can be maintained, and they account for the very different results obtained on overhead, underground and submarine circuits respectively.

Overhead wires, for many years almost exclusively used for telegraphy, and even now the only conductors that serve satisfactorily for really long-distance telephony, possess the advantage of low resistance, which can be diminished to any reasonable degree by the use of copper, and by increasing the size of the conductor; their electrostatic capacity is small, say from 0·013 to 0·014 microfarad per mile; and their self-induction is relatively great. Their use admits of attaining the maximum speed in telegraphy; and perhaps the most important limiting factor in this case is the self-induction of the receiving apparatus, which would be a useful factor if distributed over the line, but which tends to act as a choking coil when massed at the extremity. This self-induction in the case of the Wheatstone receiver amounts to as much as one henry, and its effect in prolonging the current, combined with that due to the capacity of the line, is neutralized to some extent by inserting a shunted condenser between the instrument and the earth. As already stated, this admits of a maximum speed of 450 words per minute—a relatively low figure compared with that attained by the electro-chemical receiver.

Overhead telegraphs are subject to serious interruptions, due to the falling of trees during heavy storms, and to breakdowns arising from a combination of sleety snow and gales of wind against which no engineering precautions are of any avail. On the other hand, until recently, underground wires, owing to their capacity, which reached 0·3 microfarad per mile when insulated with gutta percha, and the great cost of the material, were only used in special cases where strategical or other imperative reasons rendered such a course necessary. The introduction of dry paper as an insulating material profoundly modified the situation. The electrostatic capacity of suitable paper is not very much greater than that of air, and wires lapped with paper, cabled, thoroughly dried and enclosed in a lead sheath are not only available for long underground telegraphs, but it is not too much to say that without them the modern development of the telephone would have been wholly impossible. The Post Office has within the last few years laid underground 138,000 miles of paper-insulated wire for telegraph purposes alone, and to illustrate the bearing of this material on telephone progress it may be added that, between 1900 and 1902, before a single subscriber had been secured, the Post Office laid 240,000 miles of wire underground as the first step in the development of the London system, and from its central exchange in Carter Lane 210 ducts radiated to various portions of the centre to be served from that point; and although at that period 36,000 wires only were led into the building, the spare duct capacity admitted, as the system grew, of increasing this number to 125,000 or 150,000 wires. The mileage of underground wire laid by the Post Office in Great Britain for telephones is now increased to 521,000 miles. By no other known means could such a service have been provided, and these facts emphasize the connection between paper insulation and telephone development.

Submarine telegraphy is not susceptible of the many developments that have been possible with land-lines, the high electrostatic capacity, varying between 0·3 and 0·4 microfarad per mile, and the very long lengths that are necessary to connect the great continents of the world, rendering the use of any but the most delicate apparatus impossible on long cables. The receiving instruments originally invented by the late Lord Kelvin, then Sir William Thomson, are still the only apparatus available for the reception of messages on long transcontinental cables, and so far it has not been found possible to materially increase the speed of working, except, of course, by increasing the dimensions and cost proportionately.

Attention has been devoted to eliminating disturbing elements and perfecting the signals by the use of automatic transmission and by the use of curbing devices at the receiving end. The introduction of repeaters at intermediate points where cables land at suitable localities has recently become possible, and it may be that methods of distributed induction referred to later may ultimately be of some value, but this subject is not ripe for consideration. Short cables present no difficulties that cannot be overcome for relatively rapid working, but their electrostatic capacity limits materially the distance over which telephone working is possible, as I will show presently.

In ordinary telegraphy, when transmitting through an overhead line, the frequency of the current alternations is only 180 per second for 450 words per minute, and the current has actuated the apparatus at the further end before the battery connection has ceased. Another condition, however, is introduced when a conductor is used for telephonic speech in which a maximum frequency of 1,800 to 2,000 vibrations per second has to be dealt with. In these cases the transmission from the telephone assumes complex wave-forms, and the effect of even a moderate capacity becomes far more marked than in the case of telegraphic transmission. If a simple impulse, of the form of a sine wave, were emitted in a circuit containing neither capacity nor inductance it would maintain its form, and it would only lose in amplitude owing to the waste of energy in heating the conductor. With much capacity in the circuit, however, the wave tends to elongate, and if the capacity be sufficiently great and the line sufficiently long, the following wave overtakes the lagging tail of the previous one; they then blend more or less together, and having lost their distinctive character, fail to impress on the receiving telephone the distinct character of the sound from which they emanate. Of course, if this happens with a simple sine wave it will be obvious how much this damping action is intensified in the complicated wave-forms which speech assumes.

Of course the loss of the overtones means diminution of the timbre of the voice; in other words, though speech may still be possible, the voice ultimately becomes less recognizable until, when a certain limit is passed, if the resistance of the conductor be not too great, it may degenerate into a low-toned imitation of speech, or with the smaller conductors which are used for city work the attenuation rapidly lowers the volume of sound until it becomes unrecognizable. A few years ago I had, in conjunction with the Engineer-in-Chief of the National Telephone Company, Mr. Gill, to fix certain standards of speech which had to be maintained by the two administrations working the telephone system in this country, and after a series of practical tests, which were found to accord very closely with theoretical calculations, we found the results, given in the Table on p. 255, to represent the practical limits of ordinary speech on various classes of conductor, both overhead and underground, when no special steps were taken by artificial means to overcome the deleterious effects of capacity.

I have, however, referred to self-induction as the analogue of inertia in mechanics, and obviously, therefore, if it were possible to endow the circuit in which a wave was in movement with sufficient self-induction to prevent the tailing and consequent distortion of its form, the limit of speech would be materially increased, as only the attenuation due to resistance would have to be provided for. Now it is possible to achieve this result to a certain extent by adding artificially to the self-induction of telephone cables, technically termed loading. The ideal method would be to increase the self-induction uniformly throughout, and attempts have been made to effect this by lapping a copper conductor with thin iron wire or tape of a high magnetic permeability. Another method consists in distributing magnetic coils at uniform distances of a mile or two apart throughout the length of the line. Under these conditions the distance over which speech is possible has been increased from two-and-a-half to three-and-a-half times. The British Post Office has recently laid a cable with distributed inductance between England and France which will increase the range of speech about four times as compared with a similar type of unloaded cable.

The rapid and enormous development of the telephone service which has taken place throughout the world within the last few years is a remarkable achievement of the electrical engineer. The principle of the microphone which converts sound vibrations into the electrical vibrations already referred to, and of the telephone which reconverts the electrical into sound vibrations, are so well known that I need not dwell on the subject further than to point out that Graham Bell's telephone, as it left his hands in 1876, is at the present time essentially the same instrument, slightly improved in mechanical construction, as when he gave it to the world, but all the other adjuncts of a complete telephone service have been profoundly modified and we are not yet in sight of finality.

The problem of connecting two subscribers' lines at an exchange is a very simple one when a small number of subscribers only are concerned. An electrical indicator of the shutter type to announce

TABLE OF TRANSMISSION EQUIVALENTS BASED ON ATTENUATION.

Type of Line. (Conductor weight per mile.)	Constants per Mile of Loop.			No. of Miles of various Types of Conductors giving Equal Transmission to 1 Mile of 20-lb. Cable.	
	R. Ohms.	C. M.*f*.	L. Henrys.	Calculated Miles.	Observed Miles.
Underground Paper-Insulated Cable.					
10 lb. cable . . .	175·64	0·07	0·001	0·61	0·61
20 ,, ,, . . .	86·0	0·055	0·001	1·0	1·0
40 ,, ,, . . .	42·0	0·056	0·001	1·47	1·47
70 ,, ,, . . .	25·0	0·063	0·001	1·83	1·83
100 ,, ,, . . .	17·0	0·058	0·001	2·45	2·45
150 ,, ,, . . .	11·7	0·065	0·001	2·95	2·85
200 ,, ,, . . .	8·75	0·07	0·001	3·5	
200 ,, bunched cable	4·5	0·0846	0·001	2·8	2·8
400 ,, ,, ,,	2·25	0·108	..	..	3·5
Submarine Cable.					
160 lb. copper and 300 ,, G. P. per knot	12·9	0·12	0·00165	2·3	2·0
G. P. Quad.					
40 lb. copper per mile	43·0	0·128	0·001	0·95	0·9
Aerial Wires.					
40 lb. bronze open .	90·0	0·0075	0·00419	2·83	
70 ,, ,, ,, .	52·0	0·0079	0·00419	3·9	
100 ,, copper open .	18·0	0·00808	0·0039	8·45	
150 ,, ,, ,, .	11·9	0·00839	0·00376	11·7	11·0
200 ,, ,, ,, .	9·0	0·00862	0·00366	14·7	14·5
300 ,, ,, ,, .	6·0	0·00893	0 003545	21·0	21·0
400 ,, ,, ,, .	4·5	0·00919	0·00344	26·1	25·0
600 ,, ,, ,, .	2·97	0·00958	0·00331	36·8	36·8
800 ,, ,, ,, .	2·25	0·00987	0·00322	45·8	
400 ,, iron open . .	60·0[1]	0·01	0·012	4·1	3·7

a call, flexible conductors to connect any two lines by means of plugs and spring jacks, electrical bells to call up subscribers, and the equipment is complete. The problem is a serious one, however, when, as is now the case, the number of telephone subscribers in a large city runs into five or frequently six figures; when the area covered, like that in London, amounts to 640 square miles; when it has to be served by 60 to 100 separate exchanges, between all of which intercommunication must be provided with the minimum amount of delay, and when an exacting public expects to be connected with any subscriber on a large system immediately the call is made. Add to this the communications between all the principal cities in the country, and the fact that practically the major part of the business of the 24 hours is concentrated in about 2½ hours in the morning and a similar period in the afternoon, and the difficulties to be solved become faintly apparent, even to the uninitiated.

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[1] High resistance due to impedance at average frequency of speech oscillations.

Apart from minor improvements in the calling indicators and other details, each of which added its quota to the gradual perfection of the service, perhaps the most marked advance made, when exchanges grew in size, was the introduction of the so-called multiple switch. Before its introduction, each operator had the care of, say, 100 subscribers whose lines terminated in front of her. She could connect any two of these 100 directly, but as soon as a connection had to be made with another section a subsidiary junction line had to be brought into use and the services of another operator invoked to complete the connections; and as the number of subscribers on the exchange increased, the number of junction lines interconnecting the whole of the positions increased in a geometrical ratio. This became a very serious element in cost, and the complications and consequent delay in completing the connections between subscribers on various positions gradually increased in a much greater proportion than the number of subscribers. To obviate this difficulty the so-called multiple switch was designed, and it provided that any operator responsible for answering the calls of, say, 100 subscribers could connect any one of these directly with any other subscriber on the same exchange, even if the number reached 10,000 or upwards. To make this clear, it is advisable to repeat that a subscriber's line terminates on a small compound spring called a jack, and if a flexible metallic cord terminating in two plugs is provided, on the insertion of one of the plugs into one subscriber's jack and the other in a second jack both subscribers find themselves in direct communication with one another. In the multiple board the lines of, say, 100 subscribers terminate as before stated on one operator's position; but on a large panel above these local jacks the whole of the 10,000 or more lines are connected with a series of jacks within reach of the operator, and this is repeated at every operator's section, so that a line on any position can be directly connected with any of the other lines on the exchange by the operator who answers the call. This improvement, great as it was, did not meet all the difficulties. The shutters of the indicators had at first to be restored by hand after a call had been answered, every such operation causing some slight delay, which, multiplied by thousands of operations, eat into much time. An arrangement was, therefore, designed so that the insertion of a plug into a jack, the first operation in answering a call, caused the shutter to be restored to its normal position automatically. This, however, left untouched another source of delay, that necessary to connect mentally the particular shutter that had fallen with its corresponding jack before inserting the answering plug, which, if not correctly done, resulted in the plug being inserted in the wrong jack. Again, the close of the conversation had to be indicated by the subscribers ringing off by means of their calling apparatus, an operation often forgotten by a busy, careless or uninstructed user. If, therefore, the connection between the subscribers remained for an unreasonable time, or if another subscriber desired to speak to one of those supposed to be engaged, the operator had to come into circuit, listen for a moment, then, hearing nothing, ask "Have you finished?" before severing the connection. To obviate all these sources of delay the "common battery system" was designed. In this, the form of manual exchange equipment now considered the most satisfactory, the shutter indicators are replaced by glow-lamps which, being very small and occupying no more space than the jacks themselves, are placed immediately over the latter. When a lamp lights it indicates a call and the operator, without doubt, hesitation or delay, forthwith inserts the answering plug into the jack. At the subscriber's office no special calling apparatus is needed. The lifting of the telephone from the hook automatically lights the lamp. When the operator has ascertained, by touching the metallic rim outside the subscriber's jack with her connecting plug, that the subscriber who is wanted is free this plug is inserted, and its insertion is followed by the glowing of a supervisory lamp near the plug seat, there being two of these to each set of cords and plugs. The operator then gives a brief ring and when the called subscriber takes the telephone off the hook to answer, this is evidenced by the supervisory lamp being extinguished. The operator has no further concern with the connected lines until both subscribers restore their telephones, when both the supervisory lamps glow, the connection is severed, the lamps darken and the normal condition is restored. The work of the operator is thus conducted with absolute certainty; no intervention to ask questions is necessary, the glowing or extinguishing of the supervisory lamps indicates whether the subscribers are talking or have finished, and all her speech on the line can be reduced to "Number, please," and perhaps "Subscriber engaged" or "Subscriber don't answer."

This simplicity in manipulation is, however, as might be assumed, arrived at by considerable complication of apparatus, which if described in detail would take up a whole evening. (Views were shown on the screen of exchanges equipped by the Western Electric Company, and the British Insulated and Helsby Cables Company.) I have to thank the latter company very heartily for the working models exhibited here to-night.

The electrical energy in common battery systems is, as may be inferred from the name, provided by accumulators at the exchange instead of by primary batteries at each subscriber's office, and not only is the cost of the energy infinitely small as compared with the old system of providing separate batteries for each subscriber, but the further advantage follows that the full energy is always maintained and available on every circuit, a condition almost impossible to fulfil even under the best organization when thousands of primary cells scattered over a great city have to be looked after by linemen, some of whom may do their duty less conscientiously than others, whilst some subscribers may exhaust their batteries by careless use quicker than should be the case. Similar automatic arrangements are in use on the junction lines connecting the various exchanges necessary for the effective service of large cities.

In the period during which the ordinary telephone equipment has undergone the modifications just described, inventors have turned their attention to the design and perfection of an automatic telephone in which each subscriber, by a simple method of manipulation, may without the intervention of an operator at the exchange obtain direct access to any other subscriber connected with the service. One of the earliest systems of this type was known as the Strowger, and by the courtesy of the Automatic Electric Company of Chicago, the owners of the patent rights, and of the British Insulator and Helsby Company of Prescot, I am able to illustrate the working of the system. Each subscriber's line terminates on a line switch which forms part of a group of 100 switches. This switch is connected by ten circuits, the equivalent of the plugs and cords in a manual board, with a series of selectors each accommodating 100 junction lines. According to the size of the exchange, there may be two or three sets of selectors connected similarly by means of junction circuits, and finally, there is a connector, a somewhat similar instrument, which makes the connection between the two subscribers. Each telephone has a dial with finger holes and numbers. On removing the telephone from the hook the procedure is as follows: If, say, No. 4852 is wanted the caller inserts his finger in hole 4 and revolves the dial up to the limiting stop. This actuates the line switch, which causes a connecting plug to enter the springs of the first disengaged junction leading to the selector group of 4000. The same action follows in sequence with 8, 5 and 2, the final movement of the connector making the connection if the required subscriber is disengaged. If he be engaged, a busy back signal (a vibratory current) is given. When the connection is made and the conversation is complete the hanging up of the telephones restores the connections to the normal. A full description of the apparatus would take up a whole evening, but its working can be inspected at the close of my address. This method has had considerable development amongst the independent telephone companies in America.

On the subject of the future growth of telegraphs and telephones, few of those acquainted with the subject would venture to dogmatize, but certain statistics I have prepared will convey to you possibilities far more pregnant than any amount of speculation. The following figures, for the years 1902 and 1907 respectively, have been gathered from authentic returns, and they embrace information from every country of importance throughout the world.

TELEGRAPH AND TELEPHONE STATISTICS—WIRE MILEAGE.

	1902.	1907.	Increase.
TELEGRAPHS.			
Land wires	3,659,659	5,038,981	1,379,322
Submarine cables . . .	212,894	259,000	46,106 = 21·6%
TELEPHONES.			
Wire	7,467,417	19,839,537	12,372,120
Subscribers' stations . . .	3,534,036	8,406,336	4,873,330

Large figures frequently fail to impress the mind, but when it is stated that this mileage of wire will soon equal (if it has not already equalled) one-third the distance from the earth to the sun, the remarkable activity of the modern telegraph and telephone service will perhaps be more forcibly realized.

It will be observed that although the development of the telephone service is taking place with giant strides, telegraphs, both land and submarine, have shown remarkable growth in the intervening period of 5 years, notwithstanding the keen competition of the telephone. The great increase in the latter service, however, has taken place within a relatively limited period of time.

As regards the American Bell systems, the great advance commenced in 1896 when the associated companies only possessed 300,000 telephones. In 1905 this number had risen to 2,000,000, and in 1907 to 3,000,000, whilst at the beginning of the present year the corresponding figures are 5,100,000.

My somewhat brief and hasty generalization of the subject, necessarily limited by time, has not admitted even of reference to many subjects of interest, such as combined telegraph and telephone working on the same wires, or the use of repeaters to increase the speed of working on long lines; nor have I been able to refer to other aspects of telegraphy such as modern fire-alarm systems, railway signalling, etc., to which the public owe much in the shape of immunity from danger and accidents; but I trust that what I have said will show that this, one of the earliest branches of electrical engineering, is not stagnant, but that, on the other hand, it is very much alive, is served by some of the keenest and most advanced electricians and administrators, and is absorbing annually a large amount of capital which it is turning to good and profitable use.

The time that has necessarily been taken up in dealing, even in outline, with the preceding subjects has left little space for wireless telegraphy, but I am told that under no circumstances must I pass it by without mention. The idea of telegraphing without wires goes back to the early days of telegraph working, and methods were designed, based partly on earth-induction and partly on electromagnetic induction, which admitted of communication over limited distances. Even at the present date there are two existing examples of this method in the United Kingdom, one a wireless telegraph service between Rathlin Island and the north of Ireland, a distance of 8 miles, and the second, a wireless telephone between the Skerries Island and Holyhead, a distance of 3 miles. The range attainable, however, was so small that, when Marconi introduced a system based on the work of Hertz and other notable investigators, further attempts to improve the older method ceased.

It was known by Henry, after whom the unit of self-induction has been named, that the discharge of a Leyden jar, which to the eye appeared as a single spark, was really of an oscillatory character, and Hertz proved that when a powerful induction-coil was connected with two conductors separated by a spark-gap, each time a spark occurred the oscillations set up in the conductors caused electric waves to traverse the ether. He analyzed these waves by interposing a metallic ring, provided with a microscopic spark-gap in their path, the passage of the waves being evidenced by a spark at the gap. This detector was not fitted for telegraphic use, but the subsequent invention by Branly of a more effective detector, the coherer, paved the way for Marconi's work.

Marconi in the first instance connected one terminal of an induction coil with a lofty vertical conductor and the other terminal to earth. Each discharge across the gap resulted in electrical oscillations in the vertical conductor, and these oscillations caused electrical waves to radiate through space. As these waves cut across other vertical conductors electrical currents were generated therein which acted on the coherers, and signals were recorded. These waves were, however, highly damped, and practically only the first rapid rise had any material effect on the receiving apparatus, so that under these conditions all stations within range were affected by the signals. Further, as these oscillations had a frequency of the order of a million per second and the impulses from the induction coil were of the order of ten per second, there were relatively very long intervals between each successive wave emission. The attention of all those interested was therefore directed towards the discovery of selective apparatus by tuning, and to effect this object endeavours were made to replace the highly damped waves by as near an approach to undamped waves as possible. This was partially effected by establishing an oscillating circuit consisting of a capacity and an inductance around the spark gap of the induction-coil, so that the oscillations would be more continuous than in the first case, the vertical conductor being connected either directly or inductively with the oscillating circuit: thus the first measure of success by selective tuned circuits was achieved. So long, however, as ordinary induction-coils with their slow make-and-break arrangements were in use, the relatively long idle period between successive oscillations still existed, and the ideal system of absolutely undamped waves remained unattained.

A further step in advance was achieved by Poulsen, who replaced the spark-gap by an arc, and thus obtained an enormously increased number of primary impulses, so that the emitted waves were more and more continuous, and this method has been followed by that designed by Lepel in which the arc is replaced by two or more metal disks separated by a minute space. By substituting a telephone transmitter for the ordinary telegraphic key, it has been found possible to transmit actual speech a distance of 30 or 40 miles, whilst the limit of telegraphy may now be placed at perhaps 5,000 miles.

This very brief sketch must necessarily fail to do justice to the manifold improvements that have been introduced into all the details of working, either in spark or arc telegraphy. Each fills at present a definite sphere, and we have by no means arrived at finality in this interesting branch of the earliest type of electrical engineering.

I have to express my obligations to the Postmaster-General for permission to exhibit many of the instruments on view, to the Engineer-in-Chief of the Post Office, and to several of his able assistants who have fitted up the apparatus and have aided me in illustrating my remarks.

The PRESIDENT, in proposing a vote of thanks to the Lecturer, said the members had heard, he might almost say, an electrical review of the marvellous complication and development in telegraphic and telephonic enterprise. He would ask Sir John Wolfe Barry to second the proposal, and before sitting down he should like to say how pleased the members were to have the Postmaster-General present with them. He had no doubt that from the review which Sir John Gavey had given of his subject, there was one reflection which must have been brought home to the Postmaster-General, namely, how much those great enterprises depended on the electrical engineer and on the technical gentlemen who had served the Post Office and the country so well; and further, that he must expect to get rid of renewals and to buy new machines at a rate perhaps greater in his Department than in most other establishments.

Sir JOHN WOLFE BARRY said he had the greatest pleasure in seconding the vote of thanks to the lecturer. He was sure all must feel deeply grateful to him for the immense care he had taken to try and make plain to them some portion of the complicated work which was constantly being undertaken by the Post Office and other bodies. Associated as he was with some of the principal cable companies, Sir John Wolfe Barry felt a deep personal interest in the lecture. He recollected some 10 years ago, in an address which he gave to the Society of Arts, prophesying that within a measurable time a man would no more think of being without a telephone in his house than of not having a front-door bell. That state of affairs had not yet arrived, but the figures showing the increase in the use of telephones given by the lecturer seemed to indicate that his prophecy—he had been careful not to give the exact time—would be realized before very many years were over. He thought nobody could doubt that as this use increased the Post Office would be found equal to the demand, and that charmingly simple apparatus known as the switchboard would become somewhat complicated. The present switchboard might be very simple if one only knew how to use it, but to the uninitiated it even now presented an appalling idea of what would happen if anything went wrong. He hoped on some future occasion that they might hear a little more of cable telegraphy over the whole world, because that was a matter which was attracting much public attention, due to the natural demand for further links between our brethren beyond the sea and the Mother country. He also hoped that before very long no passenger-ship would be without the means of communication by wireless telegraphy. He could not help thinking that oversea wireless communication had now arrived at such a point of perfection that it would not be improper to make it compulsory upon passenger-ships to have a wireless installation on board. He had heard that day that the Republic of Brazil had undertaken to place at the disposal of every ship passing its great length of highly important coast the means of wireless communication with shore stations to be forthwith erected. Great Britain had already taken a good step in the same direction under the auspices of the Post Office, and the colonies were following that example. That fact had been brought before him in his official capacity on several occasions, and it was to be hoped that within measurable time every passenger-ship at sea would be able to communicate with the land in case of necessity. With those few remarks he again assured the lecturer that The Institution was most grateful to him for the pains and trouble he had taken in preparing and delivering the lecture.

Mr. Herbert Samuel, M.P., Postmaster-General, said, as the President had been good enough to refer to him, he might be allowed to join in the thanks which had been tendered to Sir John Gavey, and at the same time to express his own thanks to The Institution for having permitted him to attend the lecture. Sir John Gavey had had a somewhat difficult task, for he had not to be too simple for the expert nor too technical for those who were not experts: if he had been too simple he would have earned the derision of the expert; if he had been too technical he might have secured the somnolence of those who were not experts. Speaking as one who fell into the latter category—although, he supposed, officially he was presumed to know everything about the subject—he had been kept very much awake by Sir John Gavey's interesting descriptions of the latest developments in telegraphy and telephony. It might interest those present to know that the experiments in wireless telegraphy which had been conducted by the Post Office had been exceedingly satisfactory. The stations round the coast which had been purchased last year by the Post Office had proved most useful, so much so that the number of messages received and sent had doubled within the last 3 months. If the growth of wireless telegraphy continued at such a pace ingenious automatic sending- and recording-instruments would probably have to be provided in order to cope with the work. The only saddening reflection which occurred to him, as one who was responsible for the finances of the Post Office, when watching the marvellous developments which were taking place and hearing Sir John Gavey airily talking of the introduction of automatic telephone exchanges, was the question how long it would be before all the appliances which had cost the nation millions of money would have to be thrown on the scrap-heap and replaced by something new. Happily in the Telephone Department a sinking-fund had been established to cover expenditure within a very short period of years. He hoped that period, short though it was, might not prove to be too long by reason of the development of those later appliances. In conclusion, he must say that he felt, after hearing the lecture, that the Post Office and the country were indeed fortunate in having in its service experts of the rank of the engineer who had been good enough to address The Institution that evening.

The motion was carried by acclamation.

Sir JOHN GAVEY, in reply, expressed himself highly honoured by the manner in which the vote of thanks had been accorded him. He had felt throughout in dealing with the subject that he had had to omit reference to many points and to slur over others upon which he might with advantage have enlarged, but the limits of time had obliged him in delivering his lecture to pass over certain sections, including the important subject of submarine telegraphy, which it would be seen when the lecture was printed had been considered in some of its aspects. He thanked the members very heartily for their reception of his lecture.

SECT. II.—OTHER SELECTED PAPERS.

(*Paper No. 3842.*)

"Walney Bridge, Barrow-in-Furness."

By ARTHUR TREWBY, M.A., Assoc. M. Inst. C.E.

THE island of Walney is about 9 miles long, but not more than a mile wide, and it lies on the west side of Barrow-in-Furness in a direction pointing north west. It is separated from the mainland by the Walney channel, the sea entering from the north at the mouth of the Duddon and on the south from Morecambe bay. Since Messrs. Vickers, Sons and Maxim established their ship-building and engineering works at Barrow in 1897, a small town has sprung up on Walney island called Vickerstown, which now contains a population of some 5,000 people. A steam ferry was the only means of communication, except a ford at low tide, between the island and the mainland; and it was felt, that in view of the development of Vickerstown, some better connection was needed. After several schemes had been considered, a Bill for bridging the channel was introduced into Parliament in 1904, and passed the same year, the principal alteration made by the Committee being that the proposed opening span of 100 feet was increased to 120 feet. The contract was let in 1905, and work was commenced in the autumn of that year.

Description of the Work.—The bridge consists of eight fixed spans ranging from 117 feet 10 inches to 82 feet 8 inches clear waterway between the piers, and one opening span of 120 feet clear waterway between the timber dolphins. The total length of the bridge is 1,125 feet between the masonry abutments, measured along the centre line; the Walney abutment not being placed square to the bridge owing to the angle which the approaches make with the bridge. The fixed spans and opening span are all carried on piers, each consisting of two cylinders filled with concrete and connected at the top by a capsill girder. The opening span is on the Scherzer rolling-lift principle, in two leaves, and is worked by electricity. The width between the parapets is 50 feet, of which 31 feet 4 inches is taken up by the roadway, leaving two footways of 9 feet 4 inches on each side. Two lines of tram-rails are laid on the roadway of 4-foot gauge and 10 feet 6 inches between centres of lines.

The Scherzer rolling-lift system requires some explanation, as its introduction into this country is of comparatively recent date, and up to the present only four bridges have been completed embodying this design, although more than one hundred have been built in America and other countries. The first of these bridges to be built in this country was that over the Swale, for the South Eastern and Chatham Railway Company, of 60 feet clear span. The second is in Ireland, and was built over the River Suir, near Waterford, for the Fishguard and Rosslare Railways and Harbour Company, and is of 50 feet clear span. The third is at Walney, and the fourth is at Barrow-in-Furness over the new 100-foot passage between the Ramsden and Buccleuch docks. The motion of one of these bridges when opening is similar to that of a rocking chair, as it rolls back at the same time as the end rises. This rolling motion, besides reducing the friction which is met with in the case of an ordinary central-shaft bascule bridge, allows the same width of opening to be obtained with a smaller angle of rotation. The angle through which it is necessary to turn the bridge varies between about 70° and 80°, depending on the radius of the rolling segment, in place of an angle of 90° in the case of the ordinary bascule. It has the further advantage that the centre of gravity of the balance weight can be so arranged that the leaf is balanced at an angle of 35° or 40°, so that when opening there is an upward force assisting the motors and a retarding one as the leaf passes the midway position. In closing, the reverse takes place; and this assists materially in reducing the power, and shortening the time required to open and close compared with the ordinary type of bascule.

With the exception of the Walney bridge, all the above-mentioned bridges are single-leaf and embody the earlier designs of the Scherzer Bridge Company, which includes what are known as operating struts, moved backwards and forwards by means of a pinion engaging with the rack on the underside of the strut. The pinions and machinery operating them are placed on the adjacent fixed span or abutment, as the case may be, so as to be clear of the bridge when opened. The operating struts may be likened to a pair of shafts connected with the centre of rotation of the rocking chair, which, on being pulled backward or pushed forward, cause the chair to rise or fall. In the case of the Walney bridge no operating strut is required; what corresponds to it is a stationary rack attached to the pier and gearing with a rotating pinion placed at the centre of the rolling segment. The machinery for actuating the pinion is placed, not on the fixed span as formerly, but is attached to the underside of the opening span, the electric power being supplied by a flexible cable. In this way very much less room is taken up by the machinery, the only part that is seen being the pinions and racks; the weight of the motors and gearing also assist in balancing the leaf.

Foundations.—From several borings taken along the site of the bridge it was apparent that the foundations would rest in stiff clay as regards the majority of the piers, and in gravel towards the Walney side. The clay slopes away towards Walney, being found about the level of ordnance datum at Pier No. 1, numbering from the Barrow side, some 23 feet below ordnance datum at Pier No. 4, about 38 feet below at Pier No. 7, and 50 feet below at Pier No. 10. The ordnance datum line is the mean between high- and low-water level, and the tide rises and falls some 14 feet above and below it at spring tides.

Piers.—The cylinders for the piers are of four sizes, viz., 8, 9, 10 and 12 feet in diameter at the top, enlarging to 12, 13, 14 and 18 feet 6 inches in diameter at the bottom. The first three sized cylinders carry the fixed spans, and the last sized cylinder the opening span. As all these cylinders are of the same type it will be sufficient to describe one of them, i.e., the cylinder 10 feet in diameter which was used for six piers. This cylinder is constructed of steel for the lower portion of 18 feet and of cast iron above. The steel portion is divided into three parts; at the bottom there is a cylindrical portion 14 feet in diameter and 8 feet high composed of steel plates ½ inch thick overlapping one another. Three rings of angle-iron stiffen the lower 4 feet, and there is a shelf-plate 12 inches by ⅜ inch at a height 4 feet from the bottom. The cutting-edge plate is 9 inches by ⅝ inch. Above this cylindrical portion there is a truncated cone 8 feet high which reduces the diameter to 10 feet 3 inches, and above this there is 2 feet of cylinder 10 feet 3 inches in diameter, making a total height of 18 feet of steelwork. The bottom flange of the cast-iron cylinder is bolted to the top flange of an angle-bar ring riveted on the inside of the steel cylinder a little below the top of the steel shell-plate: the plate stands above the angle-bar ring and allows space for a ¾-inch rust-joint between the inner face of the shell-plate and the outer face of the cast-iron, and thus makes an efficient connection between the steel and the cast-iron. At the top of the steelwork there is a diaphragm-plate to which is bolted the air-shaft 3 feet 6 inches in diameter; thus all the air-pressure comes on the steelwork and none on the cast-iron rings. A cast-iron cap is placed on the top of the cylinders, and about 2 feet below it there is a moulding cast on the cylinder. The height of the cast-iron rings is 5 feet, with the exception of the closing lengths, which of course varied. All the rings up to and including those 10 feet in diameter, were cast in one piece, but the 12-feet rings were in four segments bolted together. The thickness of the metal is 1½ inch for the 12-foot rings and 1¼ inch for the others.

Where the tide allowed it the cylinders were pitched in position on the ground and built up there; but where the water was deep, they were built up on a staging and lowered by means of three hydraulic jacks with hollow rams through which the lowering links passed. These links were formed of flat steel bars, alternately double and single, and were connected at the bottom with brackets riveted to the sides of the cylinder. There were slots in the links corresponding to the travel of the ram. Before the excavation, the cylinders were partly filled with a ring of concrete starting from the shelf plate and continuing up to the diaphragm plate, leaving a working-chamber the full diameter of the cylinder for 4 feet up and after that a conical air-space. The excavation was, as far as possible, commenced in the open; but compressed air had to be used before much progress could be made. The clay, which was the principal material excavated, was in all cases fairly tough, and in places interspersed with boulders, the rate of excavation for this material being about forty skip-loads per working shift of 9 hours and the same by night. The maximum number of loads got out in one shift was 130 of fine sand, about three loads to the yard, at Pier No. 10. The cylinders were sunk until either hard boulder clay or ballast was reached, and varied but little from the depth shown on the contract drawings, with the exception of the pier nearest Walney abutment. The greatest depth at which any cylinder was founded was 30 feet below the bed of the channel, or nearly 62 feet below high water of spring tides. The maximum air-pressure used was about 27 lbs. per square inch, but no trouble was experienced with the men, only one case of "bends" being recorded, the man having had them before in compressed-air work, and being too old to continue work of that nature. The depth to which the cylinders were sunk was determined by sinking until a satisfactory foundation was obtained and the top of the steelwork was about 10 feet below the bed of the channel; when this depth was reached, the cylinder was filled with 4-to-1 concrete for a depth of 4 feet from the bottom, and then with 6-to-1 concrete to just below the diaphragm plate. All this was done under compressed air, and the pressure was maintained for about 24 hours, when it was allowed to die off. The air-shaft was then unbolted and removed, and the cylinder was filled with 6-to-1 concrete in the open. No weights were used to force the cylinders down; but concrete was put in round the air-shaft to add to the weight.

As the estimated load on the foundations was a maximum of about 5¼ tons per square foot, a test load of 25 per cent. more, or 6½ tons per square foot, was used in order to ensure that there should be no settlement during the erection of the superstructure or afterwards. The load used consisted of rails, these being laid on girders, which in turn were bedded on 12-inch timbers fitted inside the cast-iron rings and resting on a level bed of ballast. The height of the timbers and ballast was so arranged that the girders could be wedged up from under the top of the cast iron, thus throwing some weight on the cylindrical rings. This, however, was not done until a considerable weight had come direct on the concrete. The greatest load applied to any one cylinder was 1,360 tons, or 6½ tons per square foot, and the maximum settlement was about ½ inch; in many cases no settlement at all could be detected. The full test load remained on for at least seven days.

The cylinders are connected together at the top by a heavy capsill girder embedded in the concrete, which is at this point composed of 4-to-1 concrete, this proportion continuing 3 feet below the capsill girder to act in place of a stone bearing-block. The capsill girders have two webs 1 inch thick, which are 3 feet 6 inches apart and 3 feet deep. The webs are stiffened with angles and diaphragms under the bearings, the whole being filled up solid with concrete well rammed into all the corners. The top plate of the capsill girder is 1 or 2 inches above the cap of the cylinder, which allows the top to be sloped outwards, the whole being coated with 1 inch of asphalt and rendered waterproof. Four holding-down bolts are used, 12 feet long by 1½ inch in diameter. The making-up lengths for the cylinders were inserted just above low-water level, and so cast that they brought the upper portion of the cylinder to the correct position if it was only slightly out, or half-way to it in the worst cases, the remainder of the adjustment being made in the capsill girders and bearings.

Abutments.—The Barrow abutment carries the toll-houses and is about 87 feet in length at the concrete foundation, which is carried on creosoted pitch-pine piles 12 inches square, the piles being spaced 3 feet to 3 feet 6 inches between centres where they carry the bridge proper, and about 4 feet between centres under the toll-houses. The foundation was on hard clay and the piles were driven until the last six blows from a 25-cwt. hammer falling 6 feet did not cause a greater settlement than 1½ inch. The average length of the piles was 15 feet, and they were cut off a foot above the foundation, which was formed of a 6-to-1 concrete 3 feet thick.

It was anticipated that the Walney abutment would cause more trouble than that at Barrow, as the adjacent pair of cylinders had to be sunk 10 feet lower than the contract drawings allowed for before a satisfactory foundation was reached; and the result of the preliminary excavation bore this out. At the proposed level for founding the abutment, in place of gravel a sort of running sand was encountered, being a mixture of sand and clayey material. A series of seven bore-holes showed this to exist in varying thicknesses over the whole area, but at some feet below it there was a bed of fairly coarse gravel. It was therefore decided to found the abutment on this gravel and to extend and strengthen that part of the foundation under the main girders. To do this effectually it was necessary to drive a watertight dam. This was composed of timbers 12 inches by 6 inches, bolted and dogged together in sets of three, with felt between the joints, the outside timbers being cut with a V. The piles were 32 feet long, and, allowing for the joint, 2 feet 9 inches wide by 6 inches thick, and they were driven about 20 feet into the ground by means of a wide piling-hammer cast specially to drive the three piles at one time. The dam was of irregular shape to suit the outline of the abutment, but no difficulty was found in making it watertight. Clay was dumped on the outside and rammed against the timber, and the joints were caulked with oakum. No king-piles were used, but three sets of walings, 12 inches by 6 inches, were fastened on both sides with the usual system of struts. Chains acting as ties also helped to make the dam rigid and to prevent any movement due to the flow of the high tides. The top of the dam was about 2 feet above the high water of spring tides, at which time the head of water above the lowest foundation was about 20 feet. Two sluices were put in as low as possible, and a pulsometer-pump 3½ inches in diameter sufficed to keep the water down.

The original surface of the ground was about 7½ feet above ordnance datum, and when the excavation had passed through the strata of clayey sand and reached the bed of gravel, at a depth of 4½ feet below ordnance datum, it was found that at high tide the water came up through the ground and formed a number of small blow-holes. The bearing piles, which were driven 3 feet to 3 feet 6 inches apart between centres, stopped this to some extent; but it was considered advisable before putting in the concrete to spread a layer of cement bags over the foundation to prevent the water bubbling up and disturbing the concrete. The excavation was taken out in three sections, that is to say, the dam was not divided up in any way but one part of the foundation was always in advance of the next part, steps being left in the concrete to make a bond.

One of the principal reasons for spreading the foundation lay in the fact that no satisfactory test could be obtained from pile-driving. Two test piles were driven, the longer one being 43 feet in length. At a point about 21 feet into the ground it sunk 1½ inch for six blows of a 25-cwt. hammer falling 6 feet; but this set gradually increased and then diminished slightly, till when the pile was 43 feet into the ground it sunk 2⅞ inches for the same test. The borings indicated that there was a bed of coarse gravel at the point where the test pile went 1½ inch for six blows; so that it was decided to stop the bearing piles in this gravel. These piles were, therefore, about 14 feet long, and were allowed to project, as in the other abutment, 1 foot into the concrete. For the last six blows they went from 3 inches to 4 inches. Taking the worst condition of loading, and assuming that no load is taken by the ground, for that part of the wall under the bearings the load per pile is about 16 tons, this being the mean, as the centre of pressure is very close to the centre of gravity of the wall at this point. The wall under the bearings is 21 feet wide, and projects beyond the line of the wall proper some 6 feet. The whole of this portion is strengthened by the insertion of 15 inches by 6 inches rolled steel joists, 20 feet long, placed transversely to the wall, with 6-inch joists in three alternate layers above. The concrete for the first foot in the foundation of the main wall was put in at 4 to 1, and for the rest of the foundation 6 to 1. Where the sump had been, and at any part which it was difficult to drain thoroughly, 4 to 1 concrete was put in bags and well rammed in. To strengthen the foundation further, it was decided to cut the sheeting piles off at ground level, and carry the concrete hard up against them.

The foundations for the wing-walls call for little comment. The dams used here were made of two half timbers dogged together, making one pile 2 feet by 6 inches, with plain joints which proved tight enough, as the foundation-level was about 8 feet higher than in the main wall. Bearing piles were used but were spaced farther apart, and the sheeting piles were also left in. Weep-holes are provided in the main wall but not in the wing-walls, as all the water drains towards the main wall. The back of the Walney abutment is covered with one thickness of Callender's sheeting, the bottom being fastened down with rammed clay, sloping from the wing-walls downwards to the main wall so as to drain the water towards the main wall. The Barrow abutment, in place of the sheeting, was plastered with 1 inch of 2-to-1 cement mortar. Both abutments had 1 foot of rubble backing behind them.

The stone used in facing the abutments came from the neighbouring quarry of Newton, and is a very good white limestone. The facing starts about 18 inches below the level of the ground, and is rock-faced, with beds and joints dressed, up to the level of the steelwork. The toll-houses and pilasters are hammer-dressed, and the string-courses chisel-dressed. Barrow abutment carries the toll-houses, which are built of stone with a slate roof. The basement is used as a lavatory on one side, and as a store-house on the other. The upper floor is fitted up as an office with fireplace, electric light, telephone to the operating-cabin, etc. A pair of gates across the roadway, and side gates across the footways, are placed between the toll-houses to control the traffic.

Superstructure.—The superstructure is composed of two main girders of the **N** type, with cross girders and longitudinal floor beams carrying jack arches of concrete. The level of the Walney approach-road being 6 feet lower than the Barrow approach it was not possible to make the girders all of one depth, as a headroom of 6 feet above the high-water mark of spring tides had to be kept. Consequently the main girders are of three different depths; but the distance, 11 feet 6 inches, between the posts is kept uniform to simplify the floor. There are five spans having eleven bays, each 11 feet 6 inches long, with main girders 14 feet deep over angles, one span with ten bays and main girders 12 feet 6 inches deep, and two spans with eight bays and main girders 10 feet deep. These girders do not call for any special mention; the booms are 17 inches deep, the lower booms being open, with angles 4½ inches by 4½ inches by ⅝ inch and webs ¾ inch thick. The main girders are 31 feet apart between centres, and rest on cast-iron fixed bearings at one end and on rolled-steel rockers at the other. The rockers are turned to 8 inches diameter, and they are 4 inches wide; the weight on them per lineal inch, with the maximum loading possible, is about 26 cwt. The cross girders are 3 feet deep, and come just under the bottom angle of the top boom, so that the top of the longitudinal girders, which are 15 inches deep and of built-up section, is some 3 inches below the top of the main girders. At the ends of the main girders, the cross girders are raised so as to be just below the level of the top angles of the main girders, and an expansion plate connects the ends of the two adjacent spans, being riveted to the end girder of the fixed span and free to slide over the adjacent girder. The longitudinal floor-beams are seven in number and spaced to suit the tram-lines. The thickness of the concrete at the crown of the jack arches is 8 inches; and before any concrete was put on the shuttering the latter was coated with an inch of cement mortar and allowed to set. This formed a smooth surface when the shuttering was removed, and it was then limewashed. The surface of the concrete was finished off with an inch of cement rendering, and on this 4-inch prismatic creosoted oak blocks were laid. Expansion joints were placed in the oak blocks every 40 feet, and they were made of one thickness of patent compressible packing. An inch expansion joint was also left in the concrete over the piers and the space filled with bitumen. Two conduits were made in the concrete for electrical cables, one along the centre for the arc-lamps, and the other and larger one on the north side, for general supply purposes, etc. Cast-iron gullies are let into the concrete for drainage, and bolted to the main girders. The curbs are of granite and the joint between them and the roadway was made with puddled clay. The footways are carried on brackets, these supporting a longitudinal channel bar, on the top of which are small rolled steel joists, 3 inches square. The area is thus divided into spaces about 2 feet 6 inches by 3 feet 8 inches, which are filled in with 6-to-1 concrete reinforced with expanded metal. A layer of cement mortar was first put on the shuttering, and then the expanded metal was bedded in it, and the concrete, which is 3 inches thick, shovelled on. The surface is made of 2 inches of tar-macadam. This, as well as the wood paving and granite curbs, was laid by the Corporation. The parapets are of the closed type 4 feet 6 inches in height, made of steel plates with inside cast-iron skirtings and outside cornices. A moulding runs along the top, and the joint at each bracket is covered by a small pilaster and panel on the inside, a larger pilaster and panel being used over the piers, where expansion is provided for. The parapet between each bracket is divided into three by half-round vertical bars on the inside and small channels on the outside.

Cross bracing is introduced between the main girders over the piers, and at intervals between them. It consists of a bottom member attached to the lower boom of the main girders, with bracing members joining the cross girders. Cast-iron screens are placed round the bearings to shelter them, and to hide in some measure the difference in height of the main girders.

Opening Span.—The opening span is carried when closed by two pairs of cylinders 12 feet in diameter at the top, opening out to 18 feet 6 inches at the botton. A longitudinal capsill girder passes from these back to the adjacent cylinders 25 feet away carrying the fixed spans. This girder is 33 feet 9 inches long and 5 feet 6 inches deep, and it carries the track girders along which the bascules roll. As the capsill girders would be immersed in water, at high tides, for the greater part of their height, they are surrounded by box girders to within 1 foot of the top and filled with concrete and waterproofed. The track girder is 16 feet 11 inches long and 6 feet deep. It has a single web of two ½-inch plates, and the top flange is composed of 8-inch by 6-inch by ¾-inch angles and a series of six side plates 14 inches, 21 inches and 27½ inches deep by ¾ inch thick. The top is planed to give bearing to the track plates, which are of malleable steel 2 inches thick, the double row of teeth or studs projecting a further 1⅞ inch. The total length of roll is 14 feet 10 inches. The track girders are braced together by two girders 6 feet deep, with lattice webs, and cross-bracing top and bottom.

There are two main girders for each bascule, composed of a single web with 8-inch by 8-inch by ¾-inch angles and flange plates 20 inches wide top and bottom. The depth of the girders ranges from 5 feet at the centre to 15 feet 3 inches over the centre of rotation. The web and flanges are greatly strengthened at this point by a series of side plates and stiffeners. The radius of the 20-inch by 2-inch curved plate riveted to the rolling segment is 11 feet 6 inches; and it is machined on both sides, so as to fit close to the rolling segment, which was also machined to receive it. Recesses are slotted through the plate 1 foot 7 inches apart "staggered" for the studs on the track girder. The rear or tail end of the girder is 6 feet 2 inches deep, and has at the end a bearing plate which comes in contact with the underside of the bumper block seating on the fixed span when the bridge is closed. There is also a seating or bearing against which the pawls of the rear lock act so as to prevent any deflection of the rear end under passing loads.

The anchorage for the opening span is taken by the adjacent fixed span down the inclined end ties to the foundation bolts. The weight of the fixed span is sufficient to counterbalance the upward pull brought on it by loading the opening span with the assumed full live load, but provision is also made for double this amount by three foundation bolts, 3 inches in diameter, in each cylinder. The bolts are of high tensile steel, each 30 feet long, the anchor plate at their base being a circular casting 4 feet in diameter. At the top the bolts are securely fastened to the gusset plates, and before the nuts were screwed down the bolts were stressed up to 6 tons per square inch by means of hydraulic jacks. This permanent tension was given in order to prevent any lengthening of the bolts should a pull come on them. At the top of the inclined ties are gussets connecting them with the top boom, and between them is placed the steel bumper block in a stirrup formed of a bent plate riveted to the gussets. The tail end of the main girders for the opening span is narrowed so as to pass between these ties with 2 or 3 inches of clearance on each side.

The arrangements for locking the opening span at the centre are simple and effective. On the Barrow leaf are formed, at the end of the main girders, two projections, an upper and lower one, from the extension of the web and top and bottom angles. The lower projection is longer than the top one, and they form, as it were, a pair of jaws with the lower protruding. The Walney girders, on the other hand, are cut square with the lower side bevelled off. Instead of the single web continuing to the end, double webs are formed beyond the last cross girder, that is to say, for a distance of some 3 feet 5 inches, and across the end of these webs is placed a cast-steel diaphragm projecting in the form of a nose. As the leaves are closing, the Barrow one stops automatically at a certain point when nearly closed, until the Walney nose end has entered the jaws of the Barrow leaf, when they close together. The nose of the Walney leaf slides along the upper surface of the lower jaw of the Barrow leaf, and at the same time this lower jaw passes between the two webs of the Walney leaf. There is only a slight clearance between the upper surface of the nose and the lower surface of the top jaw of the Barrow leaf, so that the two leaves are locked in all directions. One leaf cannot deflect without the other; and there is a total absence of hammering at the centre under the passage of a heavy load, the rolling action of the leaves giving sufficient forward movement to allow the Walney end to bolt itself into the jaws of the Barrow end.

The cross girders are placed 15 feet apart, with the footway brackets 7 feet 6 inches apart. The cross girders are hog-backed to suit the camber of the roadway, and increase in depth as the main girders deepen. The floor for the roadway is composed of steel joists 15 inches deep with steel floor-plates $\frac{3}{8}$ inch thick laid on the top. The latter are coated with two thicknesses of pure bitumen brushed on while hot and dusted with cement. On the top of this, where the tram-rails were laid, tarred felt was placed; but the remainder of the roadway was laid with creosoted pitch-pine planks, covered with 2 inches of diagonal karri wood planking 4 inches wide. The joints between the pitch-pine planks were run with bitumen, and also the bolt-holes and the space between the planks and the rails.

On either side of the centre of the rolling segment are two special cross girders 10 feet deep, between which the machinery is placed. From the rear cross girder, which is 9 feet from the centre of the segment, longitudinal girders are cantilevered out dividing the roadway into five boxes open at the bottom, into which the counterweight is put. The last cross girder is thus some 10 feet 6 inches from the end of the span, so as to allow the counterbalance chamber to pass between the capsill girders when the bridge is opening. At the same time, the rear ends of the main girders pass between the webs of the capsill girders, clearance being left in the concrete with which the latter are filled. An oak bumper block is placed against the end of the track girder on each side, and bolted to the top of the capsill girder to act as a stop; but the leaves are always brought to rest before reaching the block.

The travel of the leaves when opening makes it necessary to cantilever out the adjacent fixed spans in order to give clearance to the balance weight. In the present case the roadway was cantilevered out from the last cross girder, some 9 feet 6 inches back, and was supported on brackets. The space between the ends of the fixed and opening spans and at the centre of the latter was covered over by grooved steel plates fastened to the fixed spans in the former case, and to the Walney leaf in the latter case. The footway is carried by longitudinal channel-bars, to which are bolted 6-inch by 3-inch pitch-pine runners, and over these 3-inch creosoted pitch-pine planking is laid. The handrailing is made of gas-piping standards and rails, with vertical bars. As the handrailing approaches the fixed spans it is widened out so as to pass outside the fixed span parapet when the bridge is opening, the last bay of the fixed span parapet being also made of gas-piping to facilitate this.

Operating gear, etc.—The opening span is operated entirely from the Walney cabin, as the electric power is brought to the bridge by a main on the Walney side. There is also an emergency hand-gear which is placed on the dolphins and geared to two speeds, so as to enable two or four men to work it. The power from the winch is taken by a horizontal shaft through the track girder, and is transmitted to the first-motion shaft by means of mitred wheels and a pendant shaft which slides up and down through a square sleeve in the lower mitre-wheel, during the opening and closing of the leaf, while the top-driving mitre-wheels on the shaft follow the horizontal travel of the centre of rotation. The hand-gear is only for emergency purposes, and is brought into use by means of an ordinary clutch. For the purposes of opening and closing, two 25-brake horse-power motors are used for each leaf. They are of the ironclad multipolar type and are placed, together with the gearing, under the roadway of the opening span. They are geared to the same shaft, and transmit motion through cast-steel gearing to the pinions engaging with the racks outside the main girders on each side of the bridge. No weight is taken by these racks, as the whole weight of the opening span is on the track girders. The rotation of the pinion along the rack causes the leaves to open and close in the same way as rotating the hub of a wheel would cause it to move backwards and forwards. A powerful magnetic brake of the twin solenoid type is fitted to each leaf, the drum of the brake being keyed to the first-motion shaft. An emergency oil-brake is fitted to the Walney leaf, and a magnetic brake to the Barrow leaf. The gearing and motors are fixed to the girderwork of the opening span and turn through an angle of about 74°; the power being brought to them by a cable encased in flexible piping. At the rear end of the opening leaves and attached to the fixed span there is, as previously mentioned, an eccentric locking-pawl which bears against the seating provided at the end of the main girders of the opening span, and so prevents any deflection of the back end of the leaf under passing loads. The pawl is actuated through worm gearing by a shaft driven by a 2 brake horse-power ironclad multipolar reversing motor. Between the latter and the gear there is a magnetic clutch to obviate the danger of the former being damaged by excess of current.

The cabin on each side of the opening span is cantilevered out on brackets beyond the footways. That at Walney has a floor space of 22 feet 6 inches by 11 feet 8 inches and is painted inside with fireproof paint. When opening or closing the bridge, the operator stands facing the Barrow side, and before him is placed a set of eight levers, with room for a spare one, similar in type to those in the signal cabin of a railway. Before starting, all the levers are in the normal position, the bridge circuit is open and the magnetic brakes are on. When No. 1 lever is pulled over, the tramway circuit is opened, No. 2 lever closes the bridge circuit, Nos. 3 and 4 levers unlock the rear ends of the leaves, Nos. 5 and 6 levers raise the two leaves, and Nos. 7 and 8 levers lower the respective navigation signals for up or down channel traffic. All these levers are mechanically interlocked on Messrs. Steven's system; and, as an additional safeguard, the levers for opening and closing the bridge are automatically controlled. The operation of opening may be recapitulated as follows:—When a vessel approaches she has to make a signal if she wishes to pass through. The operator then cuts off the electric supply to the tramcars, and stops the road traffic by placing light chains across the bridge. He then opens the bridge and signals to the vessel that the passage is clear. When closing the bridge the reverse process is gone through. The bridge can be held in any position by the magnetic brakes against any wind pressure in which vessels could navigate the channel. When the current is off the bridge, the brakes are put on by a powerful steel spiral spring, and they are taken off by the twin magnets, which are designed to exert a pull of 400 lbs. each.

The method of controlling the levers for opening and closing the bridge is as follows:—An endless chain is taken on pulleys, from the centre of the operating pinions, through the floor of the cabin and passed round a drum with spiral grooving, so that the travel of the pinion along the rack causes the drum to revolve backwards and forwards as the bridge is opened and closed. Two cam drums are geared to the chain drum, and are fitted with cams so arranged that at the critical positions of opening and closing they engage with the friction rollers mounted on standards, forming the uprights of a sliding carriage, which move in and out and so regulate the controller levers through the connecting rods under the floor of the cabin. The chain drum also actuates a pointer which is fixed to a large dial and shows the exact position of the Walney leaf when opening and closing. A similar arrangement of indicating gear is placed in the Barrow cabin, the chain drum in this case operating switches, so that the opening or closing positions of the Barrow leaf are shown by means of six electric lights placed to indicate the same on a dial in the Walney cabin. One of these positions is when the Barrow leaf is nearly closed, as it has to be started a few seconds before the Walney leaf so as to wait for it when nearly closed, until the Walney nose end bears on the projecting jaw of the Barrow leaf, when both go down together. The cams are set on the drum so as to control this operation, and to ensure that in all the operations of opening and closing the power is put on and cut off at the right moment.

By means of the interlocking and automatic controlling gear, it is only necessary for one man to be employed in the cabin to carry out all operations of signalling, opening, or closing the bridge.

Both cabins are fitted with electric light, telephones, etc., the telephone in the Walney cabin being connected with the telephone exchange. The electric power is supplied from the mains of the Barrow Corporation, and is taken to the Barrow cabin through submarine armoured cables laid in a trench, some 4 feet deep, across the bed of the channel. The signal masts are of steel lattice work and are placed at the side of the cabins, access to them being obtained by a ladder leading over the parapet on to a platform, on which is placed the motor for running the marine ball up and down the mast. An emergency hand-winch is also provided for this purpose. The starting and stopping of the motor is automatically controlled from the Walney cabin. The marine ball is painted green; and at night, lights with green and red spectacles are used.

Tram rails have been laid the whole length of the bridge, connecting with the approaches. On the opening span there are two standards to each leaf, and the slack of the wire when the bridge is opening is taken up in special trolley-poles provided on the fixed spans. Each pole is designed to take up about .63 feet of wire. As the bridge opens the wire is coiled round drums provided for the purpose, which are actuated by falling weights. All the parts are very carefully insulated, bands of ambroin being used for this purpose.

Dolphins.—The lengths of the dolphins were settled by Sir William Matthews, K.C.M.G., Past-President Inst. C.E., at an arbitration between the Corporation of Barrow-in-Furness and the Furness Railway Company. They are 300 feet long, 150 feet being on the south side, and 100 feet on the north side, the remaining 50 feet surrounding the piers. The radius at the nose is 12 feet, and the dolphins widen out to 29 feet 6 inches under the bridge. They are composed entirely of Karri wood, 250 piles 14 inches square being used. These piles are over 50 feet in length, and are driven some 20 feet below low-water level of ordinary spring tides. There are the usual raking piles to give stability. Underneath the bridge, where the counterweight comes, the dolphin is cut away so as to afford clearance when the bridge is open. This part of the dolphin was therefore specially strengthened with raking piles and struts. The piles were driven by a 30-cwt. hammer, falling 3 feet; as it was found that there is less liability to split the pile if a heavy blow is given with a short drop. The walings are 12-inch by 6-inch timber and are checked into the piles, being secured by 1-inch bolts and straps. The decking is 6 inches thick at the nose-end, and 3 inches elsewhere. Five cast-iron bollards are placed on each dolphin, the bollards passing over the top of a pile and being secured to the pile with bolts and straps, the interior being grouted up solid. All wrought ironwork is galvanized. Two fairleads are fixed to each dolphin, one at each end; and there are also four electric lights on each dolphin, an arc lamp at each end, and one electric light near the racks. Access to the dolphins is obtained from the cabins down a flight of steps, and a gangway which passes under the bridge. When the bridge is open there is also a passage from one part of the dolphin to the other in front of the opening leaves. The level of the deck is 21·25 ordnance datum, or 7 feet 3 inches above high water of spring tides.

Erection.—For the purposes of erection a substantial timber staging was built, leaving an opening of 120 feet in the centre of the channel. By Act of Parliament a clear channel of 100 feet had to be maintained throughout the erection of the bridge: accordingly, the two leaves were built in a rolled-back position in their correct alignment. A 10-ton crane on each side with a 65-foot jib was sufficient to carry out the erection of the opening span, nearly all the steelwork being in place, with the exception of the last bay of floor plates, before the leaves were lowered. The last piece of steelwork on the end of each main girder was not riveted up until the leaves were lowered, but it was found that very little adjustment was necessary as ½ inch covered the error in any direction. The fixed spans were built up in place, and riveted with hydraulic riveters where possible, otherwise pneumatic riveting was adopted. The air-pressure used in these was maintained at 100 lbs. per square inch.

After the steelwork was assembled at the contractor's yard it received a coat of red lead, and, after erection at the site, one coat of red lead and two coats of dark red oxide paint; the panels of the parapet being picked out with French grey. All the iron castings were dipped in Dr. Angus Smith's patent solution, and were afterwards painted with two coats of viaduct solution; the capsill girders and underside of the floor being also coated with this solution. The total weight of steel in the superstructure amounts to 2,150 tons, of which about 365 tons was used in the opening span. In addition to this, 250 tons of steel was put into the piers and 675 tons of cast iron; and 500 tons of cast iron was used in the parapet and as counterweight. The total cost of the work including roadway and tramway lines, etc., was over £115,000.

The bridge was commenced in the autumn of 1905, and was opened on the 30th July, 1908, by the Mayoress of Barrow, but was not completed till the autumn of the same year. In September, 1908, trials were taken to ascertain the power used and the time taken to open the bridge. In the first instance three trials were taken of the two leaves opening and closing together, from the time the rear ends were unlocked. The mean of these trials was as follows:—The leaves opened in 77 seconds, using 0·5754 Board of Trade unit, at an average of 35·8 electric horse-power. The leaves closed in 96 seconds, using 0·5985 Board of Trade unit, at an average of 28·0 electric horse-power. The total for both operations took 173 seconds, and used 1·1739 Board of Trade unit. A further three trials were taken, starting from the unlocking of the leaves by the rear lock motors. In this case the mean result was as follows:—The leaves unlocked and opened in 96 seconds, using 0·66 Board of Trade unit, at an average of 33·0 electric horse-power. The leaves closed and locked in 127 seconds, using 0·83 Board of Trade unit, at an average of 31·6 electric horse-power. The total for both operations took 223 seconds, and used 1·49 Board of Trade unit.

The engineers for the bridge were the late Sir Benjamin Baker, K.C.B., K.C.M.G., Past-President Inst. C.E., and Mr. A. C. Hurtzig, M. Inst. C.E., the work being designed and erected under the supervision of their Bridge Engineer, Mr. E. M. Wood, under whom the Author worked first on the design, and afterwards as Resident Engineer on the bridge. The contractors were Messrs. Sir William Arrol and Company, Ltd., who were represented at the site, for the first part of the time by Mr. H. Cunningham, Assoc. M. Inst. C.E., and for the latter part by Mr. W. Burnside. The sub-contractors for the electrical equipment were Messrs. Crompton and Company, Ltd.

The Paper is accompanied by 8 sheets of drawings, from which the illustrations forming Plate 6 have been selected.

(*Paper No. 3882.*)

"The Design of Fishplate Rail-Joints."

By Cyril Walter Lloyd-Jones, Assoc. M. Inst. C.E.

For engineers in charge of the maintenance of permanent way it is important to know how the stiffness of a rail-joint is affected by the tension in fish-bolts, by the distance apart of the rail-ends at the joint, by lubrication of the fishing planes, and by the fit of the fishplates. To engineers responsible for the design of permanent way it is important to know how the stiffness of a joint is affected by the section of the fishplates, by their length, by the position, number and size of bolts, and by the numerous special forms of construction in use. So far as the Author is aware, this information is not easily available.

Experimental Investigation.

In August, 1904, the Author had occasion to compare experimentally, the stiffness of several types of rail-joint in connection with a modification of the standard joint on a section of the Nizam's Guaranteed State Railway (Deccan). The experimental investigation did not require elaborate apparatus, and the results were sufficiently promising to induce him to continue it in a more general manner. The tests were carried out more or less intermittently, and comprise some thousands of readings and hundreds of tests. The general conclusions are given here with the reasoning on which they are based, although for the sake of brevity the actual readings and measurements have been omitted so far as possible.

Strength and Stiffness in a Rail-Joint.—It is necessary to draw a distinction between the strength of a rail-joint and its stiffness. A joint may be considered as subject to a bending moment, M, and the transverse strength of the joint is then a perfectly definite quantity measured by the moment of resistance of the pair of fishplates $\left(p = \frac{M\,y}{I}\right)$ exactly as in the case of a beam. Given the section of a fishplate, therefore, it is quite easy to decide the bending moment which the joint will safely stand without fracture, and this value is affected neither by the amount the joint "gives" under the bending, nor by the length of the fishplates, nor by any other consideration than the moment of resistance of the section.

The stiffness of a rail-joint is the bending moment which would be required to cause unit deformation at the joint, and the stiffness, therefore, is not necessarily connected with the strength. To measure the stiffness of a rail-joint it is first necessary to find a measure for the deformation caused by a bending moment at the joint. A co-planar system of constant forces acting on a continuous rail will cause a definite curvature at each point of the rail. Suppose the rail to be sawn through, the ends to be joined by a fishplate joint, and the forces to be re-applied; the curvature of the rail, everywhere except near the joint, will be precisely the same in the second case as in the first. Near the joint the fishplates have an effect in modifying the curvature of the portions of rail touched by them, but this effect is very small in all practical joints. In the continuous unjointed rail the slope of the rail is the same on both sides close to the section taken; in the jointed rail the slope of the rail is different on each side of the joint, the two rails being inclined at an angle $2\,\theta$. The effect of a fishplate joint on the deflection of a rail, therefore, is to cause a kink at the joint, the rail-ends being inclined to one another at an angle of $2\,\theta$; the rail otherwise behaves precisely as a continuous rail. The deformation of a fishplate joint, then, under a bending moment M is measured by the angle of kink $2\,\theta$, and a measure of the stiffness is consequently M/θ. Experiments showed that the effect of a fishplate (of ordinary design) on the curvature of the portion of the rail touched by it is a negligible quantity. The influence of very heavy fishplates on the curvature of the rails-ends is referred to later. In the present Paper the stiffness of a joint is considered as measured by the ratio M/θ, the length of the fishplate is represented by $2\,l$, and the angle between the fishing planes and the horizontal by ϕ. The constants are all taken as representing a pair of fishplates.

Apparatus.—The apparatus employed for making the tests was very simple in form. Two rails about 6 feet long were connected by the fishplate joint to be tested. Each end of the jointed rail so formed was connected to the corresponding ends of a parallel standard rail by means of shackles, the two rails being then forced apart by a screw-jack placed between them and bearing on the centre of each; they were supported on knife-edges to allow freedom of movement. The bending moment was measured by reading the deflection of the standard rail, which had been carefully calibrated, and the deformation of the joint was obtained directly by measuring the angle of kink. All measurements were made by reading through a theodolite telescope on to mirrors reflecting on an ordinary levelling staff. The deflection of the standard rail was measured from a tilting mirror fixed to a datum bar (the latter resting on the pins of the shackles) and tilted by the rail as it deflected. The angle of kink was measured by means of two mirrors each rigidly fixed to the extreme end of one of the rails at the joint, the line of sight being reflected from the first mirror to the second, and from thence to the staff. Readings were also taken from two fixed mirrors in order to correct the readings for any movement of the apparatus. The apparatus was arranged so that all the readings could be taken from two theodolite telescopes placed side by side on a table. The deflection of the standard rail was measured to $\frac{1}{10{,}000}$ inch, and the angle of kink to $\frac{1}{2{,}000}$ degree. The span over which the rails deflected, i.e., the distance between shackles, was 10 feet.

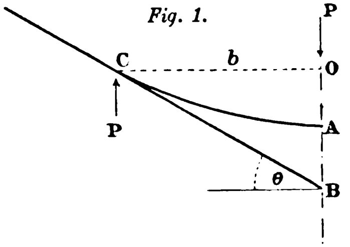
Variation of Stiffness with M.—The first tests were made to ascertain whether the deformation θ was proportional to the bending moment M; and with the exception of certain tests made with short fishplates, this was found to be so. Although the proportionality of stress and strain is a general law, it conceivably might not apply to the conditions obtaining in a fishplate joint, and it was, therefore, necessary to investigate this point. As a result of the inquiry it was determined that M/θ is independent of M, and is therefore a moment measuring the stiffness of any given joint.

Variation of Stiffness with the Clearance between Rail-ends at Joint.—In a fishplate joint a clearance is left between the butting ends of the rails; in most joints it is possible to adjust this clearance between, say, $\frac{1}{8}$ inch and 1 inch, and experiments were carried out to ascertain whether variations in the clearance, between these limits, affected the stiffness of the joint. The tests gave an entirely negative result, and it was concluded that the clearance between rail ends, varied within reasonable limits, does not affect the stiffness of the joint. This result has a bearing on the theory of the action of the joint. The first theory formulated to explain the action of the fished joint was to suppose that the fish-bolts fix the fishplates rigidly to the rails, and that the kink 2 θ is caused by the bending of the small length of fishplate between rail-ends. This is the result that would occur if there were no relative vertical movement between the rails and fishplates. The tests referred to in this paragraph demonstrate the inaccuracy of this theory, by proving that M/θ is independent of the clearance distance, whereas the theory would require M/θ to vary directly as the clearance. Also calculating from the experimentally determined value of M/θ in the cases examined, it was ascertained that on this theory it would require a free length of several inches of fishplate between the rail-ends to account for the kink which occurred. The results of these tests effectively disposed of this theory, and proved with certainty that when a fishplate joint is strained the rails and fishplate take up quite separate deflection curves. In all future investigations, therefore, it was necessary to formulate some theory to account for the relative vertical movement between fishplates and rails; the movement must consist of a sliding or other movement at the fishing planes, and is only rendered possible by reason of some secondary strain in the fishplate or bolts.

Variation of Stiffness with Bolt Tension.—The first source to look for to account for the sliding of the fishplates on the fishing planes is the fastenings, i.e., the fish-bolts. It is conceivable that some give in the bolts permits the fishplates to move outwards, and thus allows freedom for vertical movement relatively to the rail. In order to dispose of this point tests were made to ascertain whether the tension in the fish-bolts affected the stiffness of the joint. Here, again, the result was entirely negative. A pair of well-fitting fishplates were tested, taken off, refitted and retested several times, each time varying the bolt tension, and the same value was obtained for M/θ within the limits of experimental error. In the extreme cases tests were made first with bolts hand tight, without using a spanner, and then with bolts tightened as tightly as possible with a 15-inch spanner, both without alteration of the ratio. Comparative tests of this kind have been made at various times during the investigations, and the only occasions on which the bolt tension has been found to affect the stiffness were when fishplates have been used in rails for which they were not suited, i.e., when there was not a true fit in the fishing planes. The study of the fishplate joint here described does not embody such cases. The results of the tests described in this paragraph were important, inasmuch as they enabled free comparisons to be made between different joints under different conditions, although attempts were made so far as possible to obtain uniform conditions as regards bolting. The possibility of the sliding occurring by any "give" of the bolts was effectually disposed of, and it became necessary to look for some other explanation for the vertical movement relatively to the rail. A working theory was now framed by supposing the bending moment M to be transferred to the fishplate by means of a couple, P b where one force P acts at the joint o, and the other force P at some distance along the fishplate (*Fig. 1*). Then if the fishplate is bent by the couple into the curve CA, the slope of the rail will correspond with CB, and the vertical slip with AB. As θ varies directly with M, the slip AB would vary directly with M, and it might then be easily proved that $\frac{M}{\theta} = \frac{2\,EI}{b}$, which would enable the distance b to be calculated from the observed value of M/θ. This theory suggested that tests should be made to investigate the effect of varying the position of the bolts, for as the position of the forces could be calculated it certainly would appear that bolts placed in the neighbourhood of these forces would restrict the slip AB.

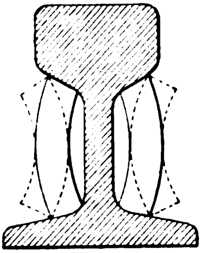
Variation of Stiffness with Position of Bolts.—These tests were now carried out with a number of fishplates of various types. Some four-hole, five-hole (central bolt), and six-hole plates of standard pattern were tested, besides fishplates drilled for special bolt-spacing. In all cases the results were entirely negative, i.e., it was found that, provided the fishplate has at least one bolt on each side of the joint, the stiffness of the joint M/θ is unaffected by either the position or number of the bolts. As an instance, a heavy, standard pattern five-hole fishplate for a 75-lb. B.H. rail was tested, and the same value of M/θ was found in each of the following cases, namely, with five bolts, omitting centre bolt, omitting centre bolt and two outer bolts, and with the centre bolt only. The conclusions arrived at were, that as the position of the bolts does not affect the slip the cause of the slip cannot be due to bending in a horizontal plane (i.e., to bulging outwards between the bolts); and also that it is unlikely that the reactions are approximately located at points as suggested, as in that case the proximity of bolts would be likely to affect the slip. On the result of these tests it was decided to develop a theory assuming that the reaction per unit length (i.e., intensity of vertical load transmitted to the fishplates by the rail at the fishing planes) is w at any point x, and

Fig. 1.



that the load w causes the bending deflection as well as the slip y. It was also provisionally assumed that the slip y is proportional to the load w causing it. The tests just described render it certain that the slip cannot occur due to bending in a horizontal plane, e.g., by the fishplates bulging outwards between bolt-holes; the only other ways in which "proportional slip" could occur are, therefore, either by compression of the fishplate or by the fishplate bulging outwards as in *Fig. 2*. At this point in the investigations it was thought that the former cause of slip would not satisfy the conditions, as it would neither account for the amount of slip that occurs nor for the generally accepted theory of increase of slip with increase of fishing angle. The latter cause—outward buckling—would seem to fulfil the conditions, and it was decided to investigate the truth of the "proportional slip" hypothesis by (1) investigating the variation of stiffness with thickness of web of fishplate, and by (2) verifying the law of variation of stiffness with length, which law is indicated by the curves in *Fig. 3*, obtained from the results of a mathematical analysis of the hypothesis. The former tests were tried first, with the following results:—

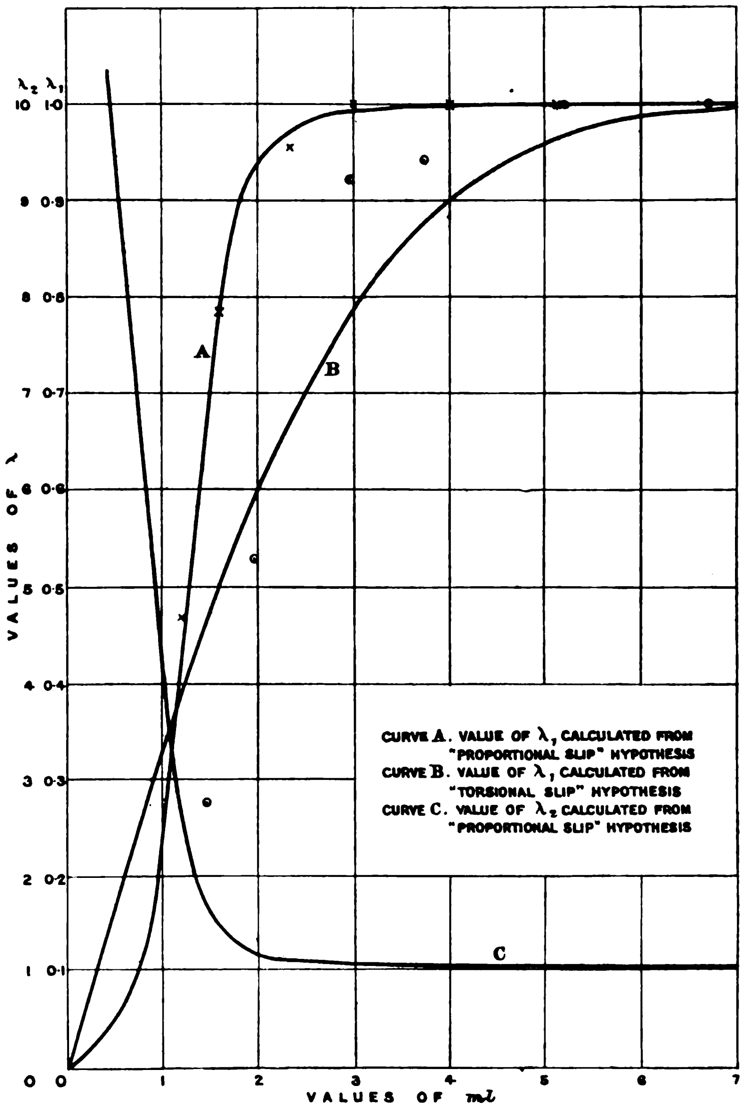
Fig. 2.



Variation of Stiffness with Web-thickness.—Four types of fishplate were tested to examine this relation. Two of the types were of angle-shaped section for flat-footed rails, and the remaining two types had projections at the foot parallel to the web to suit bull-headed rails. In all cases the procedure adopted was to test a similar pair of plates under normal conditions, and test them again after one of the pairs had had the web-thickness reduced by planing and the other pair had been reduced in section by planing off a portion of the flange or vertical projection. The amount of metal planed off was adjusted so that after planing the moment of inertia would be about the same in both pairs. In all the cases tested the reduction of the thickness of the web caused about the same reduction in stiffness as the reduction in the thickness of the flange or the reduction of the depth of the fishplate. The conclusion from these tests was that slip cannot occur by outward buckling of the fishplate.

The tests to verify the relation between stiffness and length were then undertaken, but the effect of friction became so great with short fishplates that it was decided to abandon this part of the investigation until later. The only remaining foundation for this

Fig. 3.



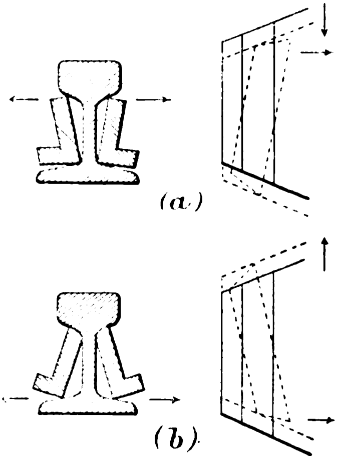
VARIATION OF STIFFNESS WITH LENGTH OF FISHPLATE.

(The plotted points were obtained experimentally. See p. 291.)

"proportional slip" hypothesis now rested on the assumption that the slip was caused by the compression of the plate. It was thought at the time to be unlikely that the slip could be explained by compression of the fishplate, principally for the reason that the magnitude of the fishing angle would not then affect the stiffness. If the slip were caused entirely by compression of the fishplates the stiffness of the joint would be independent of the fishing angle. Most of the tests made up to this point had been on rails with relatively large fishing angles (30° and 60° with the horizontal), and it was realized that with smaller fishing angles (approximating to the angle of friction) any horizontal component of the reaction would be relatively small. For the time, however, the further investigation of the proportional slip hypothesis was abandoned in order to investigate another possible effect of the horizontal components of the forces acting on the fishing plane.

Hypothesis of Torsional Slip.—It being certain that during the action of a bending moment at a joint some vertical movement does occur between rails and fishplates, it had been concluded at this stage of the investigation that such movement could be caused neither by straining the fish-bolts, by bending outwards in plan, nor by bending outwards in section (*Fig.* 2). There is, however, still another strain which the horizontal forces tend to produce in the fishplates, and this might account for the necessary relative vertical movement between them and the rail. The horizontal component of the load at the fishing planes causes a twisting moment on the fishplate, and the resulting torsional strain would permit of this vertical movement. The conditions which, according to this hypothesis, would prevail at the joint are indicated in *Fig. 4a*, the horizontal component of the pressure of the rail at the fishing planes would cause the top of the fishplates to be twisted outwards, the bottoms coming inwards, and allow the rail to move downwards relatively to the fishplate. In *Fig. 4b* is shown the corresponding effect near the ends of the fishplate; the horizontal component is outwards and situated at the

Figs. 4.



TORSIONAL SLIP.

lower fishing planes, and the vertical movement of the rail is upwards relatively to the fishplate. This "torsional slip" hypothesis was suggested by certain effects noted while conducting tests; thus several times when the load was applied to the jointed rail the fish-bolts, particularly those farthest from the joint, became slack, although they had been made quite tight beforehand; it was also noticed that they became tight again after taking off the load. This result would occur if the fishplates moved in the manner required by the present hypothesis.

To investigate the correctness of the hypothesis before comparing various joints tests were made of fishplates manufactured from steel tees, the tables being planed to fit the rails. Four pairs of fishplates were made from 4-inch by 4-inch by ½-inch tees to suit a rail with 30° fishing angle, and were then secured by a bolt at each end. After testing them the web was planed off and the fishplates were retested; then two more bolts were added and the plates were again tested. No difference was detected in the flat plates when fitted with two or four bolts, but the stiffness of the tee fishplates was roughly twice the stiffness of the same plates without the web, although the vertical moment of inertia was practically the same in both cases.

When a smaller fishing angle, however, was used, namely, 14°, the results showed no considerable difference in the stiffness of fishplates manufactured from a 4 by 4 by ½-inch tee with and without the web portion. The conclusion arrived at from the experiments was that in fishplates of small fishing angle the "slip" effect is caused entirely by the compression of the fishplate and the proportional slip hypothesis applies, but that with fishplates having a large fishing angle (e.g., of 30°) this compression may be combined with twisting, particularly if the fit of the fishplate is not quite accurate.

Investigations of the curvature of the fishplate when the slip is caused by (1) compression or (2) twisting, leads in both cases to somewhat similar results. There is introduced a constant m, which is an inverse of a length and dependent on the design of the fishplate. In the case of proportional slip (by compression of the fishplate) the value of m would be almost constant for all designs of plate; in the case of torsional slip m would vary considerably with the fishing angle and torsional strength of the fishplate. Experiments carried out on thirteen different designs of plate showed that m varied in inch units from 0·61 to 0·74 for 14° fishing angle, and from 0·31 to 0·55 for 30° fishing angle. The stiffness is given in both cases by the equation $\frac{M}{\theta} = \lambda\, m\, E\, I$, where λ is unity for all fishplates of ordinary length. The factor λ depends only on the value of *ml*, and becomes smaller as the length of the fishplate is reduced, until it vanishes when the length is very short; the form of this expression is different in the two cases, and is plotted in *Fig. 3.* This diagram affords a means of verifying the results by experimentally investigating the variation of stiffness with length.

Variation of Stiffness with Length.—These tests were commenced early in the investigation, but owing to the difficulty in obtaining consistent results they were not brought to a satisfactory conclusion until most of the other work had been completed. The difficulties were partly due to the amount of time occupied by removing, sawing, and refixing the fishplates in the machine, and also to the necessity for ensuring that the plates were not bent or twisted during the process of cutting. In some cases the fishplates were not originally true, and took quite different bearings as they were cut; these specimens were discarded; another difficulty arose over the measurements, as only light loads could be applied to the short plates owing to the high intensity of the resulting stress between the plate and rail. The effect of friction made some of the results appear inconsistent, particularly as an incorrect method was used for correcting for it when the torsional slip hypothesis had been adopted. The effect of friction was only seen in the tests of short plates, and it was eventually eliminated by smoothing and oiling the surfaces and by measuring for repeated small loadings. The results shown in the above Table are averages of three specimens of each of the two types of fishplates tested, and these figures are also plotted by the side of the curves in *Fig. 3.*

85-lb. F.F. Rail with Standard Fishplate.				66¾-lb. F.F. Rail with Standard Fishplate.			
Length. $2l$	Relative Deflection.	ml	λ	Length. $2l$	Relative Deflection.	ml	λ
Inches.				Inches.			
18	15·0	6·67	1·0	26	42	5·2	1·0
14	15·0	5·18	1·0	20	42	4·0	1·0
10	16·0	3·70	0·94	16	42	3·2	1·0
8	16·3	2·96	0·92	12	44	2·4	0·95
6	28·5	2·22	0·53	8	53	1·6	0·79
4	53·0	1·48	0·28	6	90	1·2	0·47

It has been stated elsewhere that the effect of friction at the fishing planes was found to have little effect on fishplates of the normal length, but to have considerable effect on very short fishplates. The mathematical analysis of the proportional slip shows that on that hypothesis the effect of friction would be evident with short plates, and with short plates only. The conclusion from the investigation is that the theory of proportional slip is consistent with the experimental result that the friction at the fishing planes does not affect the stiffness of a joint when the plates are not very short.

Before leaving the subject of the frictional resistance at the fishing planes it may be pointed out that whatever torsional strain occurs in fishplates having large fishing angles, it is certain that very little strain of this nature can occur in plates having fishing angles as small as 14°. The coefficient of friction of the surfaces would probably be between 0·1 and 0·2, so that the horizontal component of the vertical forces is negligible in the case of such small angles. The results showed that although there may be a torsional strain with 30° fishing angles, yet this strain has less effect in decreasing the stiffness of the joint than has the compression of the fishplate.

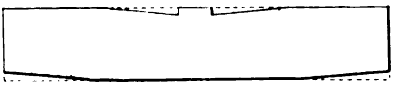
NOTES ON DESIGN OF A RAIL-JOINT.

Recapitulating the foregoing conclusions, it is clear that the design of a fishplate joint involves the consideration of its stiffness, its strength to resist the bending moment to which it is liable, and finally its strength to resist the bearing pressure at the fishing planes. The strength of a joint to withstand a bending moment M is given by the equation $z p = M$, where z is the modulus of the pair of plates $\left(\frac{I}{y}\right)$ and p is the safe stress. The stiffness of a joint is measured by $\frac{M}{\theta}$, where $\frac{M}{\theta} = \lambda_1 \, M \, E \, I$, and the intensity of bearing pressure is measured by $w = \lambda_2 \, 2 \, m^2 \, M$, where w is the maximum pressure in tons per inch length of fishplate, λ_1 and λ_2 are constants depending on the length of the fishplate, and m is a constant depending on the width of bearing.

Connections.—Neither the number and position of the bolts, the tightness with which they are screwed up, the lubrication of the fishing planes, nor the amount of clearance between the rail-ends have any effect on either the strength or stiffness of the joint. On the other hand, these considerations very largely affect the efficiency of the joint as a preventive against the track buckling, so that from this latter consideration it is important to fit as few bolts as possible and those only moderately tight. Further, the fishing planes should be kept lubricated and a full clearance allowed between the rail-ends. The liability of a track to buckle when subjected to sudden changes of temperature is a very serious trouble in some countries, and in India derailments frequently occur from this cause. Probably, in some cases, more bolts than necessary have been fitted to thin fishplates with a view to maintain a better fit, but this consideration can have no weight with the modern thick fishplate. The Author is of opinion that two bolts are sufficient fastening for any ordinary form of joint. Two bolts are certainly sufficient for the joint to do its work, and as fish-bolts never break in the road except under a spanner it seems hardly necessary to have any additional bolts as a standby.

Section of Fishplate.—The strength and stiffness of a joint are necessarily expressed in terms of the bending moment M to which the joint may be subjected; they are also proportional to the moment of inertia of the pair of fishplates, which quantity may be increased indefinitely. The bearing pressure per unit length of fishplate, however, depends solely on m, and as this constant which is proportional to the width of the bearing cannot be greatly increased, it follows that the safe bearing pressure limits the bending moment which can be applied to any joint. The Author's experience with fishplates in use is that the failure of joints is as often due to indentation of the fishing planes as to bent or fractured fishplates, and the former method of failure is just as effectual as the latter. When fishplates become indented the appearance is somewhat as shown in *Fig. 5*. The result is that the joints are down and the running becomes bad; moreover, the joints being down causes an initial kink to which is added the ordinary kink due to the running load, causing the shock due to the passage of each wheel to be increased, and consequently the stresses in the fishplate to be augmented. In extreme cases the indentation increases to about $\frac{1}{8}$ inch, when a fracture is started. It has been stated that the intensity of bearing pressure can only be reduced by increasing the width of bearing. This distance is limited by the accuracy of rolling. It seems likely that m will seldom be very much above $\frac{3}{4}$ (inch units), so that when w, the safe bearing pressure per inch length of fishplate, is settled, the safe bending moment to which the joint may be subjected can be determined from the equation $w = 2\, m^2$ M. As already explained, the joint cannot be strengthened in any way to withstand a greater

Fig. 5.



INDENTATION OF FISHPLATE.

value of M than that settled by the safe bearing pressure; an increase of the transverse strength or of the length does not affect matters. It would probably be safe to assume that w should not be greater than 23 tons per inch length. On this assumption the minimum value for the modulus of the section may be estimated,

$$M = p z \text{ and } w = 2 m^2 M, \text{ hence } z = \frac{w}{2 m^2 p}$$

where p is the safe tensile stress. Taking $w = 23$, $m = 0 \cdot 6$ and $p = 10$, the value of z is $3 \cdot 2$ in inch units, and the maximum safe value of the bending moment is 32 inch-tons. The reasoning here is that assuming that the pressure at the fishing planes should not exceed 23 tons per inch length then the bending moment acting on the joint should not be greater than 32 inch-tons, and consequently the modulus of the section to give sufficient strength need not exceed $3 \cdot 2$ in inch units. This applies to any fishplate unless it is so designed that the pressure on the fishing planes may exceed the foregoing limit. The following are particulars of the method of failure of three types of fishplates on the Nizam's Guaranteed State Railway:—

Rail.	Fishplate.	Value z.	Failure by Bending.	Failure by Indentation.
68 D.H.	Plain bar	1·47	Frequent	Never.
66½ F.F.	Angle	2·61	,,	Frequent.
75 B.H.	Protruding foot	4·49	Never	,,

The weaker fishplates are protected by special sleeper spacing. It will be seen that the fishplate with $z = 4 \cdot 49$ indents before it breaks, that with $z = 1 \cdot 47$ it breaks before it indents, and that with $z = 2 \cdot 61$ it fails in both ways. This is consistent with safe value of the modulus as $z = 3 \cdot 2$.

These conclusions require that the track shall be designed to the fishplate, and not the fishplate to the track. As a fishplate cannot be designed to withstand more than a certain definite bending moment, dependent on the fit at the fishing planes, it follows that the track must be so designed to prevent a greater bending moment from acting on the joint. The foregoing considerations only decide the safe bending moment and the minimum section for strength; the stiffness will be proportional to the moment of inertia which may be designed to suit any stiffness required. As, however, the bending moment on a rail-joint must be kept within a certain figure there seems to be little advantage in increasing the stiffness of a joint even when the weight and stiffness of the rail are increased. The stiffness should be such as to limit the angle of kink to a certain permissible value dependent on the speed of trains. The angle of kink in minutes permitted by a fishplate may be taken as $\frac{20}{I}$, where I is the moment of inertia of the fishplate in inch units. This is calculated from the foregoing formula taking—$M = 33$, $m = 3/4$ and $E = 13,000$.

Design of Length.—In the foregoing the values of the length constants λ_1 and λ_2 have been taken to be unity. An examination of *Fig. 3* will show how these constants vary with ml. The stiffness constant λ_1 is unity when ml is great and falls rapidly to zero when ml becomes less than 3. The pressure constant λ_2 is also unity when ml is great, but rapidly increases when ml is less than 3 until it becomes infinite, when ml is zero. No advantage is obtained by increasing the length of a fishplate so that ml is greater than about 3, but when the length is such that ml is much less than 3 the stiffness rapidly decreases and the pressure on the fishing planes becomes very great; when $ml = 3$ the values of the constants are $\lambda_1 = 0 \cdot 993$ and $\lambda_2 = 1 \cdot 05$. A length $ml = 3$ would correspond to $l = 4$ when $m = \frac{3}{4}$ and to $l = 6$ when $m = \frac{1}{2}$. Probably a safe length for fishplates would be 14 inches ($2l$ being the length), and as m is not very variable this length should suit all cases.

Fishing Planes.—Modern rails are rolled with flat fishing angles, usually about 14°, and the foregoing remarks on the design of fishplates more correctly apply to joints with these small angles. The effect of increasing the fishing angle is to permit twisting to take place and so to decrease the value of m, but not to a very great extent. Probably an average value for m is $0 \cdot 70$ for 14° fishing angle and $0 \cdot 60$ for 30° fishing angle. The effect of obtaining a smaller value for m is to lower the pressure on the fishing planes and so enable a higher bending moment to be carried by the joint; at the same time it requires a larger moment of inertia to obtain the same stiffness.

It is probable that a consideration of greater importance than the fishing angle is the accuracy of the fit of the fishplates and rail at the fishing planes. On this account alone different specimens of the same pattern fishplates will sometimes give considerably different values for m. The matter is of importance, as not only does it affect the stiffness of the joint, the pressure on the fishing planes and the critical length, but the width of bearing also determines the safe pressure that may be allowed at the fishing planes and therefore the bending moment the joint will sustain without failure.

The following case may be cited as an example of the effect of the fit of fishplates on their stiffness. Three types of fishplates were stocked as all suitable for a certain 68-lb. double-headed rail and all three types appeared to be equally satisfactory when bolted up. A test of the three gave the values for *m* as 0·65, 0·51 and 0·17 respectively. An accurate comparison of thin sections sawn from each fishplate with a section cut from the rail then showed the bearings to be quite different in each case. The first two had surfaces in contact; the third had contact along a line only. In carrying out tests with the 66¼-lb. flat-footed rail referred to in all the foregoing tests, it was found at first that most irregular results were obtained with various specimens of the same type of fishplate. This was eventually traced entirely to the fit of the plates, and it was then found possible to get uniform results by carefully truing up the fishplate to suit the rail. Accurate measurements were then made of the fishing angle of rail and fishplates by sawing off sections and measuring them. The following are results from two different types of fishplates and rails:—

Rail Section.	Section tested.	Top Fishing Plane.		Bottom Fishing Plane.	
		Right Side.	Left Side.	Right Side.	Left Side.
		Degrees.	Degrees.	Degrees.	Degrees.
66¼-lb. F.F. . . .	Rail	28	28	30	30
	Fishplate	24	24	33	33
68-lb. D.H. . . .	Rail	32	32	32	32
	Fishplate	31	31	33	33

It will be seen that the top and bottom fishing angles are slightly different, but that the sum of them is the same (or nearly so) for the rail as for the fishplate. This arrangement cannot be considered conducive to a good fit. The accuracy of the fit at the fishing planes is probably one of the most important elements determining the properties of the joint.

Conclusions.—Summarizing, the Author submits that the most suitable form of fishplate is one 14 inches long and secured by two bolts. There is no advantage in giving the section excessive strength or stiffness, as it is not advisable to allow it to be subject to a bending moment greater than a certain value (dependent on the safe pressure on the fishing planes), which is probably about

32 inch-tons. The moment of inertia should depend on the angle of kink which it is decided to permit, and should be determined by the formula $I = \frac{20}{\theta}$, where θ is in minutes of angle and I in inch units. The fishing planes should be oiled and ample clearance allowed between the rail-ends; the bolts should be only moderately tight. The section of the fishplate should be designed to be clear of the sleepers and sleeper fastenings. The sleepers should be spaced to prevent a bending moment greater than 32 inch-tons from coming on the joint, by moving the two adjacent sleepers as near to the joint as possible consistent with ease of packing, and by decreasing the spacing of the other sleepers near the joint.

The Author is also of opinion that weakness in a joint cannot be remedied by substituting a heavier or stiffer fishplate unless the weak fishplate fails by bending without any signs of indentation. The remedy is to reduce the bending moment to which the joint is subject either by adding sleepers, respacing the sleepers, or using a heavier rail.

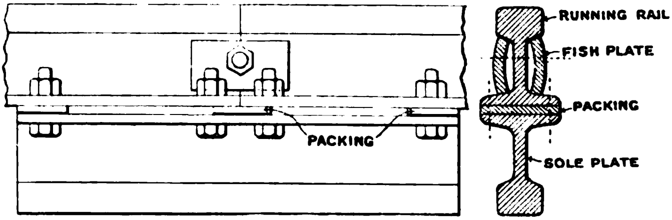
SOLE-PLATES.

The fact that a fishplated joint cannot be designed to take a bending moment greater than a certain limiting value makes it necessary to consider whether the bending moment coming on the joint can always be kept down to that limit. The Author's estimation of the limit as 32 inch-tons is based on his experience of the three sections of rails referred to on p. 294. No doubt a more extended observation of the behaviour of joints would enable this limit to be fixed more accurately. As axle-loads and train speeds increase there will be greater and greater difficulty in maintaining the road in such a state that the bending moment at the joint is sufficiently low. There is a limit to the close spacing of sleepers at joints, and in most roads this has already been reached. Such being the case, it will probably be necessary eventually to discard the fishplate and adopt some other form of joint. In this connection the action of a stiff joint-plate placed under the rail (*Fig. 6*) is of interest. This form of joint is used extensively with tramway rails, but has not been adopted with railway rails in recent years. It would be necessary still to use a very short fishplate, say 4 inches long, to connect the ends of the rail and take up the shear; the sole-plate might be quite short, and only connected to the rail at the centre and at the ends. Packing would be necessary to separate the sole-plate from the rail, as a bearing between the joint and the end of the sole-plate would reduce the stiffness of the joint. It can be shown simply from first principles that the stiffness of such a joint would be—

$$\frac{M}{\theta} = \frac{3}{b} \frac{1}{\frac{1}{I_F} - \frac{1}{2 I_R}} E$$

where b is half the length of the sole-plate, and I_F, I_R are the moments of inertia of the sole-plate and rail respectively. This compares with the formula $\frac{M}{\theta} = m \, I \, E$ for an ordinary fishplate. The distance b depends on the tension it is desirable to put on the end connections and the pressure to be allowed at the joint, the tension at the ends being $\frac{M}{b}$, and at the centre $\frac{2 M}{b}$. It will be noticed that when $I_F = 2 \, I_R$, there is no kink at the joint, and if the sole-plate were designed to that stiffness, and the length $2 \, b$ were arranged to suit the strength of the connections of the sole-plate to the rail, it seems possible that a satisfactory joint might be evolved. No doubt the end connections would be troublesome to design, as it is essential that they should not work loose and yet not grip the surfaces sufficiently tightly to prevent expansion. That some joint essentially different from the fished joint is necessary for increased loads and speeds seems certain.

Fig. 6.



Sole-Plate Joint.

Most of the experiments referred to in this Paper were conducted on fishplates the moments of inertia of which were small relatively to the rail—usually about one-third. The sole-plate joint would require the moment of inertia of the sole-plate to be twice that of the rail. In the case of a pair of side fishplates of the same stiffness, it would be necessary to consider the rail curvature, which would be at least comparable with that of the fishplates. In the foregoing formulas relative to fishplates the rail curvature has been neglected. The investigation of the fishplate deflection curve when the bending of the rail is taken account of, showed that the foregoing formulas may be applied with slight alteration unless the fishplate is very stiff relatively to the rail. For accurate results, however, the preceding expressions for the stiffness and pressure on the fishing planes of a fishplate joint require to be multiplied by the following factors, to allow for the curvature of the rail:—

$$\text{Stiffness factor} = (1 + r)^{\frac{1}{4}}\text{ ; pressure factor} = (1 + r)^{-\frac{1}{2}},$$

where r is the ratio of moments of inertia of fishplate to rail. It is evident that the corrections to be applied for the bending of the rail are extremely small unless r is great. The fishplates used in the tests correspond with values of r between 0·1 and 0·4, and the stiffness factor, therefore, varies between 1·02 and 1·09. It is of interest to examine the values of these factors when $r = 2$; in that case the stiffness factor is 1·31 and the pressure factor becomes 0·578, so that such a joint would be limited to a bending moment of 55 inch-tons, and would allow a kink of 1 minute of angle (taking $m = 0·6$, $I_r = 60$, and $E = 13,000$). The strength of the joint would, therefore, be only slightly increased by very considerably increasing the stiffness of the fishplates, and a kink would still occur at the joints.

It appears therefore that a sole-plate having twice the stiffness of the rail would prevent any kink at the joint, and could be designed to take any required bending moment. On the other hand, a pair of fishplates having the same moment of inertia would still give a limited strength to the joint (about 55 inch-tons), and would permit the rails to form a kink.

The Paper is accompanied by diagrams from which the Figures in the text have been prepared; there are also several mathematical Appendixes.

(*Paper No. 3832.*)

"On the Underground Water of the Guadaira Watershed, Seville."

By CHARLES ARTHUR FRIEND, M. Inst. C.E.

THE watershed of the River Guadaira lies to the south-east of Seville and has an area of 520 square miles. In shape it resembles a leaf with the stalk between the town of Alcalá de Guadaira and the River Guadalquivir, into which the Guadaira falls, a little distance below Seville (*Fig. 1*). Across the valley from north-east to south-west, 16 miles in a direct line, following a line drawn through Carmona, Viso, Mairena de Alcor, and Alcalá de Guadaira, rises a range of hills, 300 to 350 feet in height, closing it in to the west except where the Guadaira breaks through a gorge a mile or so in length close to the last-named town. From the gorge to the river mouth the stream is dammed by numerous weirs, and the water-power has been used from time immemorial for driving a number of flour mills. The hills are of a tertiary limestone called in the locality "Alcor," and by the Spanish term of "piedra caliza"; the stone is very absorbent, and furnishes abundant sources of excellent water, which have been utilized since the time of the Romans for the supply of a city older than Seville and later for Seville itself.[1]

It is the object of this Paper to record the result of observations made in 1907, and the relation of absorption and evaporation to the rainfall.

The geological formation of the watershed consists of a thick impermeable clay bed of the Miocene period, on which rests the limestone said to be Pliocene, while the valley is entirely alluvial and of recent formation. The clay, which has probably a thickness of 300 feet, slopes gently to the river-bed, outcropping on the left bank of the river in the gorge at Alcalá at about 100 feet above datum (sea-level), and also near Mairena at 330 feet where it is worked for pottery (*Figs. 2*). Along the dividing ridges which separate the Guadaira watershed from that of the River Guadalquivir to the south-west, and that of the River Corbones to the north and north-east, the level of the clay rises to over 300 feet above datum. The

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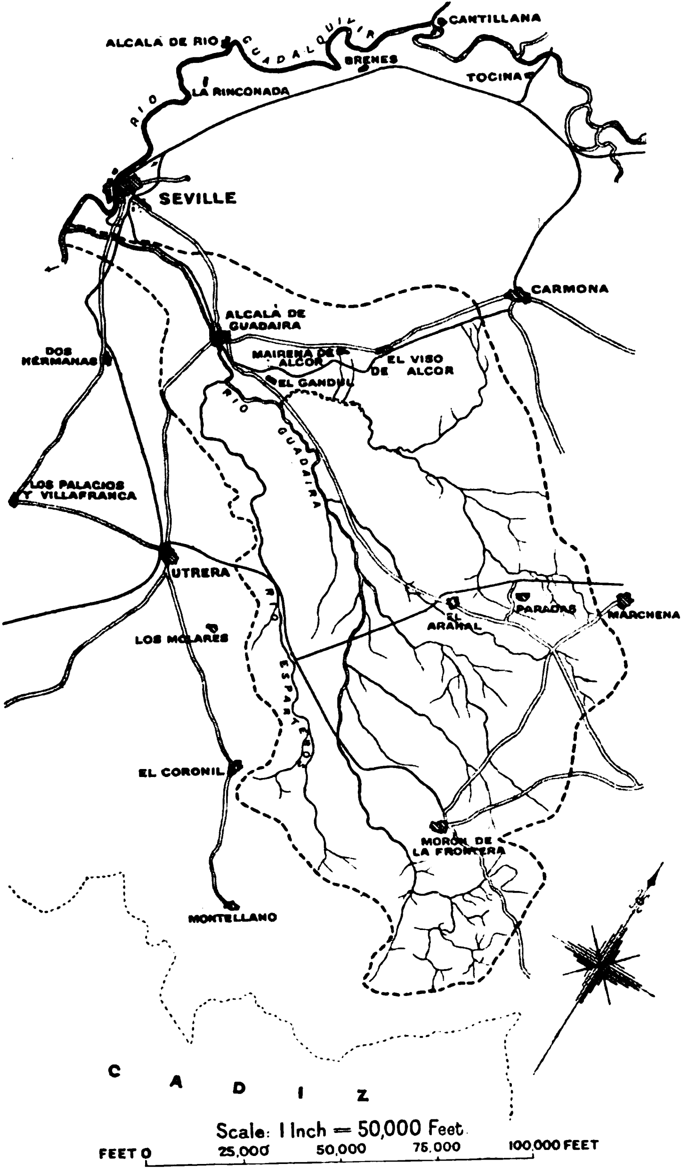
[1] Minutes of Proceedings Inst. C.E., vol. lxxviii, p. 334.

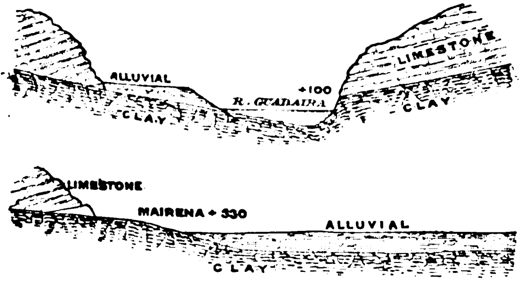
Fig. 1.


CANTILLANA
ALCALA DE RIO
RIO GUADALQUIVIR
BRENES
TOCINA
LA RINCONADA
SEVILLE
CARMONA
ALCALA DE GUADAIRA
DOS HERMANAS
MAIRENA DE ALCOR
EL VISO DE ALCOR
EL GANDUL
RIO GUADAIRA
LOS PALACIOS Y VILLAFRANCA
UTRERA
EL ARAHAL
PARADAS
MARCHENA
LOS MOLARES
RIO ESPARTEROS
EL CORONIL
MORON DE LA FRONTERA
MONTELLANO
CADIZ
Scale: 1 Inch = 50,000 Feet
FEET 0
25,000
50,000
75,000
100,000 FEET

junction of the limestone, which dips slightly towards the north, with the clay, forms the natural outlet of the underground water

The stone is largely burnt for lime, and although it is not hard enough to make good road-metal, it is so used, and the detritus forms an excellent binding material for the surface. Its power of absorption is very great: a block of dressed stone, dried and weighed, was immersed in a laboratory testing-tank until the air had been expelled and the stone saturated. On being taken out and the surface water allowed to run off, it was reweighed. The tests were made with a 0·20 metric cube in metric measurements, the water taken up being 21·75 per cent. of the volume. In British measure, taking the test piece as an 8-inch cube, it weighed 608·48 oz. when saturated, and 546·78 oz. dry, the difference, 61·70 oz., being equivalent to 106·68 cubic inches of water or 20·8 per cent. of the volume of 512 cubic inches.

Figs. 2.



The rainfall has been recorded since 1869. At first the readings were kept from December 1st of one year to November 30th in the year following, but later the natural year has been adopted. In making the analysis given below the readings of the earlier years have been adjusted so that they all begin on January 1st. The months of July, August and parts of June and September are rainless, the first rains falling in the latter month and the last in June. In any estimate, therefore, of discharge and water-supply, it will be truest to the real conditions to keep the rainfall record by harvest years, i.e., from summer to summer. The observations have accordingly been rearranged and the analysis placed in a parallel column to the other.

		Natural Years. 1870 to 1906.	Harvest Years. 1869-70 to 1905-6.
Mean annual rainfall (37 years) . . .	Inches	21	20·5
„ maximum rainfall	Per cent.	199	191
„ minimum „	„ „	30	32
„ of 3 consecutive driest years.	„ „	43	48

The mean temperature is 67·57° F. In summer the climate is almost tropical; the maximum shade temperature reached 122° F. (50° C.) in 1881, and in the 20 years from 1881 to 1900 it exceeded in every case 105° F., while in the sun, readings ranging from 125° to 140° F. were recorded. The winter is mild, but in 1883 a minimum of 21·57° F. was registered. These readings are taken from the returns issued daily from Seville University and published in the *Madrid Gazette* by the Royal Observatory.

Averages for 20 Years (1881 to 1900).	°F.
Mean annual temperature	67·5
,, maximum shade temperature	115·0 (46·1 °C.)
,, minimum temperature	31·0
,, maximum sun temperature	135·7

With the summer temperatures just described the evaporation of the surface water is enormous; the mean daily loss from this cause during July, August and September oscillating between 0·26 inch and 0·49 inch, but in the months from October to March, when the greater part of the rain falls, it will be seen from the Tables that the evaporation is practically in inverse ratio to the rainfall. From January to March the mean daily evaporation for the 20 years 1881 to 1900 was 0·08 inch, and from October to December 0·09 inch. The annual mean daily evaporation for the same period is 0·196 inch. The annual loss from this cause on the surface of a reservoir will be therefore as much as 72 inches, more than one-half of which takes place in the four summer months, June to September (*Figs. 3* and *4*).

The discharge of the River Guadaira is not perennial except for the last 9 or 10 miles of its outfall, no water reaching the river in summer above the upper weirs of the water-mills, about 1½ mile above the town of Alcalá. Within this portion of the river some water from underground sources, used exclusively for motive power, runs into the Guadaira, and there are also springs in the river-bed. Above the reach impounded by the highest weir the river-bed in summer is dry, except where stagnant pools are formed in the irregularities of the bottom.

Data derived from the Hydrographical Division of the district in the Report of the Royal Commission on Irrigation in the Guadalquivir Valley give the following information as to discharge:—

	Percentage of Discharge.
Rainfall exceeding 36 inches	19
,, ,, 28 ,,	16
,, ,, 20 ,,	15
,, ,, 16 ,,	13
,, under 16 ,,	8 to 10.

The highest coefficient of discharge recorded with 36·4 inches of rainfall is 22 per cent. in 1891–2.

The yield of the underground water first came under the Author's

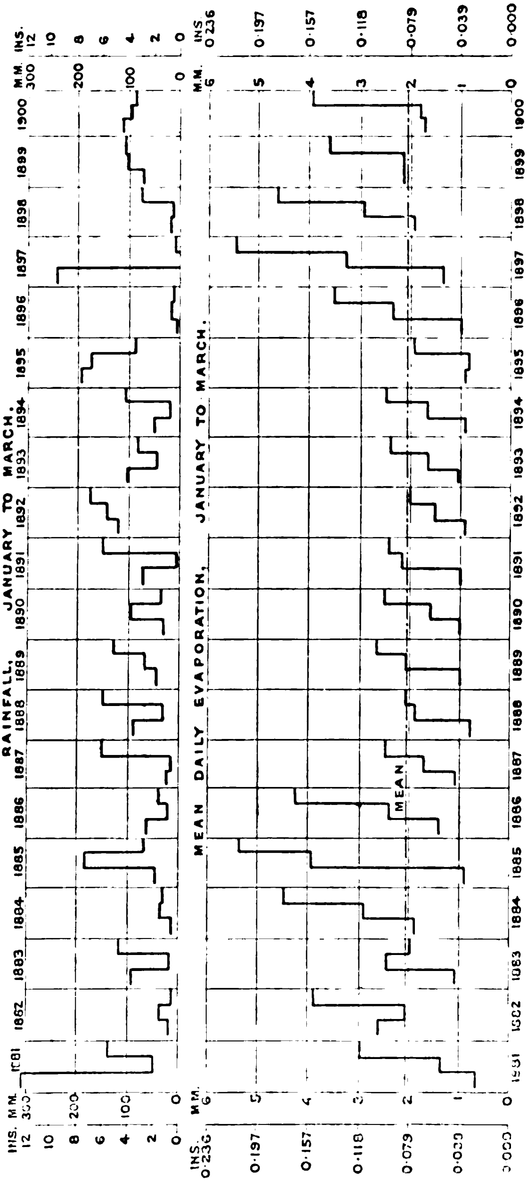


Fig. 3.

observation in 1883 in connection with the water-supply of the City

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of Seville, when periodical measurements of the sources in the Guadaira valley were instituted, which have been continued up to

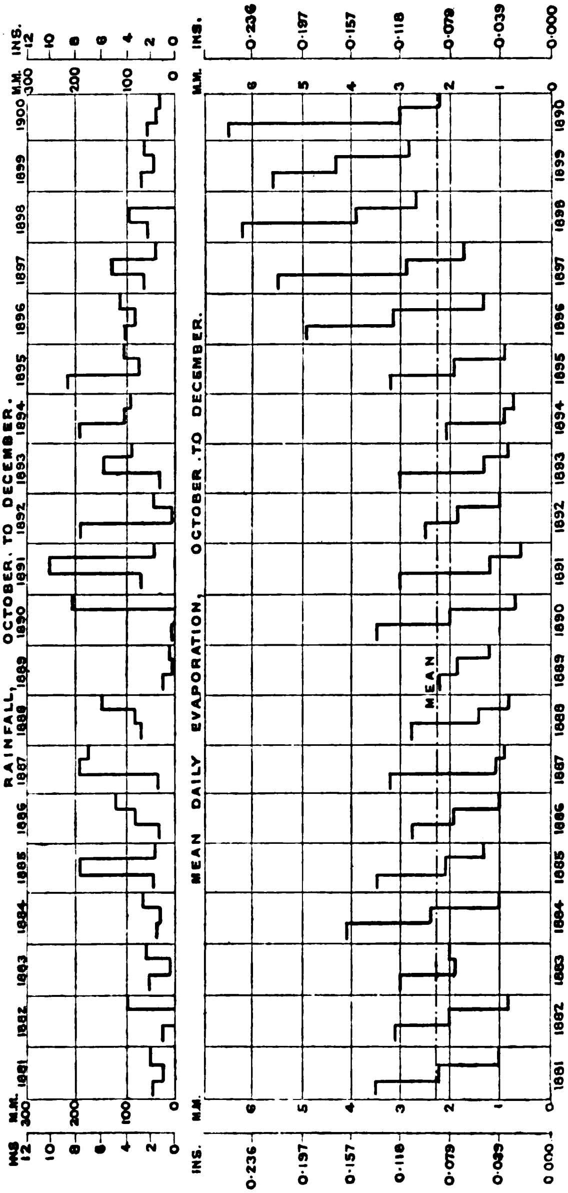

Fig. 4.

the present time at intervals of 2 months. The existing masonry mill-channels and weirs were used but without any recording instruments, and where no other means existed the water was measured by hectolitre and stop watch, the mean being taken of at least six observations on the same day. Later, when the works were built, reservoirs collected the spring water at the different sources, whence it gravitated to the pumping-station. In these reservoirs the variations of water-level are measured daily at intervals of 3 hours, and the readings entered on the pumping returns, which show the production of the sources and the amount pumped to the service reservoir. The measurements every 2 months are a check on the daily readings, and the sources are all gauged on the same day. With the exception of two wells, all the water is from springs and adits which flow out at the level of the ground. The pumps at the wells work continuously, and a daily record is kept of delivery and water-level.

The volume of underground water varies very greatly according to the rainfall, not of each year taken by itself, but according to the rainfall of groups or periods when the fall is much below or much above the mean. When maximum readings of 31 inches, 36 inches and upwards occur and recur with frequency, the underground water will yield in the different springs, wells and adits a volume altogether exceeding 11 million gallons per 24 hours on the average of the year.

In the decade 1887–8 to 1896–7, there were 3 years which gave the above average readings, and between each group containing a maximum there was only 1 year below the mean rainfall. But the volume of water diminishes very rapidly with the recurrence of years of low rainfall. Taking the 10 years from 1897–8 to 1906–7, which do not contain 1 year of maximum rainfall, and only 3 years above the mean during that period, the production of underground water gradually diminished, the level of the water in the limestone fell, and the result was that shown by the measurement of 1907 in the Table.

In the Zacatin adit, mentioned again later, it has been observed by the daily gaugings in the reservoir that in very wet years the water increases with great rapidity from an average discharge of about 550,000 gallons per 24 hours to four times that quantity (minimum flow autumn 1907, 132,000 gallons). It commences in December if 16 inches of rain has fallen since the summer, and continues to rise until the following June; it maintains a very high volume of discharge, and if the succeeding year is normal as to rainfall it will show small signs of decrease until the second summer.

This phenomenon occurred in 1887–8, 1891–2, and 1894–5, since when the rainfall has been, as has been said, low. It is attributed to the overflow of the lip of the clay ridge dividing the Otivar valley from that of the Zacatin: the construction of the Otivar adit in 1903 will probably affect its recurrence, but until a year of maximum rainfall takes place again there is no proof that it is so.

Measured on the plan, the catchment area on the right bank, reduced from metric scale to British measure, is 15,321 acres, (6,200 hectares), and on the left bank 2,965 acres (1,200 hectares), making a total of 18,286 acres. These figures have been obtained by measuring the areas bounded by the levels of the outfall of the adits and the springs, and by the ridges dividing the Guadaira watershed from those of the Guadalquivir and Corbones. At Clavinque (3) and Otivar (9) a line of borings determined in each case the point at which the ridge of the clay occurs, and which nearly coincides with the visible divide on the surface. The geological map of the district does not furnish any data which modifies the result of the measurement of this area.

In the Table is given the measurement of the yield in each case in gallons per 24 hours, taken in the autumn of 1907. This year has been selected as being the shortest both in rainfall and production since 1883, when the water was first regularly measured, and because there have been included all the known sources and not only those comprising the water-supply of Seville. The method of gauging already described, by pumping, applies to all the Seville water-supply, Nos. 3, 6 to 9, and 12 to 14, except No. 7, which is measured by a weir without recording apparatus every two months. Nos. 1, 2, 4, 5, 10 and 11, not included in the Seville water-supply, were measured in 1907 at the same time as the other sources.

In October and November of each year, the autumn rains have not yet affected the flow of underground water, which is at its lowest point, and at the same time the readings are unaffected by the losses of water due to irrigation from shallow wells by water-wheels called "norias," which are exactly the same as the Egyptian "sakir." The number of these water-wheels has been counted within the catchment area, and the average amount of water raised by them estimated from the examination of several cases which have come under the Author's notice. On the data thus obtained, an approximate estimate can be made of the water extracted from the ground by the foregoing water-wheels during the summer months, commencing in May and usually ending in September, and not less than 550,000 gallons per 24 hours should be allowed.

The yield of the sources on the right bank of the river is $\frac{3,104,000}{15,321} = 202 \cdot 6$ gallons per acre per 24 hours, and the total of both right and left banks $\frac{3,676,000}{18,286} = 201$ gallons per acre per 24 hours, or 0·37 cubic foot per second per 1,000 acres. The rainfall for 1906–7, the year under consideration, was 9·1 inches, or 44 per cent. of the mean, equal to 564 gallons per acre per diem. These figures give a value for absorption of $\frac{201,100}{564} = 35 \cdot 6$ per cent. of the rainfall. Adding the discharge into the river, including small springs and filtration which escape measurement as underground water, 10 per cent. of the rainfall, there remains 54·4 per cent. which is lost by evaporation.

TABLE OF KNOWN SOURCES OF UNDERGROUND WATER.

	Locality.	Above Sea-Level.	Yield per 24 Hours.	Derivation.	Use.
	Right Bank.		Gallons.		
1	Alcaudete . . .	524	145,000	Adit	Water-power for mills
2	Bencarron . . .	328	22,000	„	Irrigation
3	Clavinque . . .	374	550,000	Well and bore-hole	Seville water-supply
4	Gandul . . .	262	189,200	Adit	Water-power for mills
5	Marchenilla . .	229	349,800	Springs	„ „ „ „
6	Las Aceñas . .	98	528,000	Adit	Temporary supply for Seville
7	Santa Lucia . .	98	1,100,000	„	Seville water-supply
8	Zacatin . . .	141	132,000	„	„ „ „
9	Otivar	246	88,000	Adit and well	„ „ „
			3,104,000		
	Left Bank.				
10	Oromano . . .	98	44,000	Adit	Water-power for mills
11	Huerta del Moro .	82	44,000	„	Water-power for mills and irrigation
12	Arroyo del Negro.	82	88,000	„	Alcalá water-supply
13 & 14	Fuen santa and La Judia . . .	82 & 75	396,000	Adit and spring	Seville water-supply
			3,676,000		

The approximate figures of the volume of underground water in years of maximum rainfall have little or no importance in treating of water-supply, and are only submitted as lending some support to the result obtained from the Table. The average daily yield of 11,000,000 gallons per 24 hours, or 600 gallons per acre (1·1 cubic foot per second per 1,000 acres), with a rainfall of 31 to 36·5 inches (1887–8 and 1891–2) represents the absorption of 27 to 31 per cent. of the rainfall, and adding 19 per cent. discharge into the river, it leaves 50 per cent. for evaporation.

The records of the yield of underground water outside the area utilized for the Seville supply are incomplete except for the year 1907. It is hoped that they will be continued regularly in future and furnish additional data.

At present the conclusion apparently justifiable is that with a rainfall equal to that of three consecutive driest years, which is what has to be considered in dealing with questions of water-supply, the underground water is 35 per cent. of the rainfall. If the water extracted from the ground by the water-wheels is deducted as unavailable, 30 per cent. may be accepted as a safe figure. Evaporation can be taken with some certainty as 50 to 60 per cent. of the rainfall, and in impounding reservoirs 40 inches of depth should be allowed for the evaporation of surface during the summer months; a daily mean of 0·3 inch.

APPEN

TABLE I.—

(Readings reduced from
Height of Gauge above

No.	Years.	September.	October.	November.	December.	January.	February.
	1869–70	0·207	0·482	0·663	2·859	1·514	2·751
	1870–71	0·008	0·267	2·257	2·222	1·030	1·395
	1871–72	0·564	0·984	3·046	1·616	1·814	2·548
	1872–73	0·547	3·402	1·010	2·353	0·437	0·542
5	1873–74	..	1·623	1·474	0·474	0·780	0·917
	1874–75	0·015	0·719	0·353	2·089	0·622	2·355
	1875–76	0·171	1·040	0·716	4·632	0·381	2·840
	1876–77	0·476	2·038	7·680	7·718	4·382	..
	1877–78	4·648	..	1·110	1·413	0·856	2·409
10	1878–79	0·687	8·439	3·650	2·955	4·061	1·036
	1879–80	0·528	5·813	12·400	2·493	0·341	1·759
	1880–81	0·373	4·077	1·497	0·584	12·277	1·695
	1881–82	0·154	2·391	0·783	1·937	0·717	1·766
	1882–83	0·402	0·838	0·010	4·009	3·645	0·427
15	1883–84	0·052	2·332	0·291	2·356	0·337	2·119
	1884–85	0·818	1·404	1·179	2·463	2·098	7·079
	1885–86	0·941	1·463	7·303	1·645	3·034	0·759
	1886–87	1·038	1·325	2·750	4·324	0·694	0·596
	1887–88	..	0·964	7·060	6·450	3·534	1·025
20	1888–89	0·702	2·636	3·351	5·271	1·448	2·363
	1889–90	2·256	0·782	0·528	0·767	2·608	1·605
	1890–91	0·836	0·160	..	8·206	3·562	..
	1891–92	0·674	2·666	8·592	1·162	5·057	4·757
	1892–93	0·278	5·762	0·203	1·897	2·256	1·861
25	1893–94	2·524	1·105	5·671	3·307	2·057	0·591
	1894–95	0·276	7·368	3·665	3·857	7·520	7·562
	1895–96	1·811	8·586	4·452	3·504	0·315	0·630
	1896–97	..	2·392	2·609	3·599	9·043	0·118
	1897–98	..	1·941	5·468	1·504	0·692	0·681
30	1898–99	0·535	2·240	3·882	..	2·551	3·630
	1899–00	0·173	2·862	1·311	2·574	3·767	3·669
	1900–01	1·362	2·480	1·039	1·641	2·901	2·665
	1901–02	0·586	1·700	1·685	2·212	0·126	8·343
	1902–03	1·064	7·508	6·716	0·626	1·275	..
35	1903–04	0·653	1·429	0·358	3·898	1·732	1·799
	1904–05	1·944	3·412	4·098	1·350	0·889	0·011
37	1905–06	1·535	2·708	5·823	1·700	0·429	0·846
	1906–07	0·897	1·956	2·795	0·421	0·649	0·397
Monthly mean		0·782	2·685	3·092	2·686	2·406	1·987

DIX.

RAINFALL.

millimetres to British Inches.)

Sea-Level, 96 Feet.

March.	April.	May.	June.	July.	August.	Totals.	Mean.	No.
0·774	0·616	0·187	0·032	..	0·630	10·735		
2·358	0·390	1·576	0·415	..	..	11·918		
1·661	0·949	0·342	0·052	..	..	13·576		
1·869	0·918	1·177	1·772	..	..	14·027		
0·180	0·425	0·546	0·039	..	0·147	6·605		
0·908	0·877	0·773	0·238	..	..	8·949		
1·607	0·454	2·973	0·126	..	0·085	15·025		
1·893	1·252	1·957	1·180	0·148	0·098	28·822		
2·447	1·044	1·721	..	0·045	0·031	15·724		
1·808	2·055	0·084	0·529	..	..	25·304	15·066	10
1·721	1·203	4·810	0·063	..	..	31·131		
5·374	9·303	1·050	0·135	0·270	..	36·635		
0·312	0·619	1·622	0·010	0·174	..	10·485		
4·568	3·638	5·750	0·817	..	..	24·104		
1·206	3·549	0·185	0·550	..	0·010	12·987		
2·401	4·010	..	3·984	..	1·694	27·130		
1·202	1·709	0·413	..	0·100	..	18·569		
5·527	1·171	0·968	0·870	..	..	19·263		
3·881	7·135	1·022	0·035	..	..	31·106		
4·833	2·290	1·066	0·582	..	0·005	24·547	19·331	20
2·431	2·778	..	0·159	..	..	13·914		
5·468	0·603	1·050	0·572	..	..	20·457		
7·242	2·409	2·371	1·608	..	..	36·538		
2·537	2·414	1·106	..	..	..	18·314		
4·216	2·523	1·391	0·081	..	..	23·466		
3·151	3·189	1·897	0·790	..	..	39·275		
0·433	..	1·785	0·434	0·037	..	21·987		
0·149	0·984	2·523	..	..	..	21·417		
3·114	1·059	1·358	0·043	0·007	..	15·867		
4·051	0·051	0·917	1·346	0·011	0·007	19·221	20·569	30
3·099	4·027	2·330	1·425	..	0·023	25·260		
4·027	2·492	0·496	0·594	..	0·015	19·712		
2·649	5·791	2·417	0·039	0·346	0·709	26·603		
0·689	2·090	0·838	2·169	..	..	22·975		
3·279	0·645	0·311	1·094	..	..	15·198		
0·055	0·504	2·090	0·185	..	..	14·538		
2·378	1·291	0·787	1·460	..	..	18·957	20·541	37
0·063	0·452	1·177	..	..	0·330	9·137	20·248	
2·515	1·972	1·396	0·616	0·030	0·089			

TABLE II.—TEMPERATURE AND EVAPORATION.

	1881	1882	1883	1884	1885	1886	1887	1888	1889	1890	1891	1892	1893	1894	1895	1896	1897	1898	1899	1900
TEMPERATURE. ° Centigrade.																				
Max. shade .	50·0	46·2	46·0	47·0	46·8	46·8	47·0	43·2	43·0	41·4	45·2	47·5	48·5	45·0	47·0	46·4	46·8	48·0	47·6	43·8
Minimum .	−1·0	1·0	−5·8	0·8	−4·2	−0·8	2·4	−1·2	−1·4	−3·4	−3·8	−1·6	0·5	−0·6	−1·0	−1·0	0·2	1·2	2·2	1·4
Mean, 19·8° C.	20·7	20·4	19·4	20·1	19·5	20·4	19·6	19·1	18·6	19·1	19·2	19·8	20·2	19·2	19·9	19·3	20·5	19·7	20·9	19·4
Max. Sun .	60·0	59·0	58·0	58·4	59·6	56·4	59·9	56·4	55·0	55·0	58·4	51·8	59·9	58·0	57·6	58·6	57·6	59·2	58·8	55·4
MEAN DAILY EVAPORATION. Millimetres.																				
Jan. to Mar.	1·6	2·8	1·8	3·0	3·3	2·6	1·7	1·6	1·9	1·7	1·8	1·4	1·7	1·7	1·2	2·2	3·3	2·8	2·6	2·1
Apr. ,, June	5·4	6·4	4·8	6·9	9·1	9·0	5·6	5·1	4·3	5·9	5·9	..	4·9	4·8	6·2	6·0	8·8	6·8	9·0	8·1
July ,, Sept.	6·7	8·5	8·7	9·3	11·2	9·5	8·1	7·6	8·0	7·9	7·6	..	8·1	7·9	9·0	10·8	10·3	11·4	11·4	12·5
Oct. ,, Dec.	2·2	1·6	1·9	2·5	2·3	1·9	1·7	1·6	1·7	2·0	1·6	2·7	1·7	1·2	2·0	3·1	3·4	4·2	4·2	3·2
Year . . .	4·0	4·9	4·4	5·5	6·5	4·7	4·3	4·0	4·0	4·4	4·3	..	4·0	3·9	4·3	5·1	6·6	6·1	6·8	6·7

Mean 5 mm. or 0·196 ins.

(*Paper No. 3872.*)

"The Schenectady Hydro-Electric Power Works."

By Francis Ogden Blackwell, M. Inst. C.E.

Near the town of Schaghticoke, N.Y., the Hoosic River has a fall of about 150 feet, which in the past has been partially utilized by means of a series of six wooden crib-dams. These dams gave heads of from 8 to 40 feet, and furnished power to drive powder, grist, twine and woollen mills and also saw-mills. They were practically without storage to equalize the flow and were equipped with old and inefficient water-wheels.

The Schenectady Power Company was organized to build a modern hydro-electric development, to supply power to meet local requirements and to transmit the balance to Schenectady 20 miles to the west of Schaghticoke.

At Johnsonville, 5 miles above Schaghticoke, there was an old dam used to develop a fall of about 12 feet, and above this point the river-bottom widens out having little fall for 3 miles. It was found practicable to raise this dam to give a fall of 35 feet for power, and in addition a large storage capacity was obtained.

The steam power-stations of the General Electric Company in Schenectady were larger than any probable water-power development on the Hoosic River, and Schenectady therefore furnished both a market for the water-power developed and a steam reserve for periods of low water. The problem then was to obtain from the river all the power which could be generated at an annual cost sufficiently below the cost of steam operation to justify the investment in the water-power plant; and under these conditions the extreme minimum flow of the stream was less important than the useful mean flow during an average year.

Capacity of Plants.

The drainage area of the Hoosic River at Schaghticoke is 635 square miles, with a rainfall having an average for 70 years of 39·3 inches. During the low-water season each of the mills above the town utilizes the flow of the stream to meet its requirements, so that the daily flow is very variable, but the average monthly minimum is 200 cubic feet per second. The highest flood was probably about 20,000 cubic feet per second, or 31·5 cubic feet per second to each square mile of drainage area.

The average flow for the 5 years from 1904 to 1908 at the Government gauging-station at Buskirk, 9 miles above Schaghticoke, is given in the following Table in cubic feet per second. To this has been added the equivalent horse-power at 145 feet working-head and the kilowatt-hours electric output at 80 per cent. plant efficiency.

Flow.	Number of Days.	Equivalent Power.	Equivalent Electrical Output for each Increment of Flow.
Cubic Feet per Sec.		HP.	Kilowatt-Hours.
150	365	2,000	13,000,000
225	354	3,000	6,140,000
300	335	4,000	6,000,000
375	307	5,000	5,500,000
450	278	6,000	4,960,000
525	258	7,000	4,620,000
600	242	8,000	4,340,000
675	229	9,000	4,100,000
750	214	10,000	3,820,000
			52,480,000
1,125	156	15,000	15,700,000
1,500	111	20,000	11,740,000
			79,920,000

After estimating the cost of developments for various capacities, it was found that the lowest cost of power would result from building the plant for a flow of 750 cubic feet per second, which could be obtained for 7 months during an average year. This, it will be noted, is on the basis of 1·18 cubic foot per second per square mile of drainage area. As a continuous load can seldom be secured, it was assumed that the night flow would be stored for day use and the waterways and machinery were laid out for a flow of 1,500 cubic feet per second or 50 per cent. load factor; which is equivalent to the yearly output of 52,480,000 kilowatt-hours. This called for water-wheels at Schaghticoke of 20,000 HP. capacity. The pond behind the Schaghticoke dam covers 145 acres, will hold 45 million cubic feet, and will supply 750 cubic feet per second for 17 hours.

At Johnsonville it was found practicable to build a dam to store 380 million cubic feet of water, and to construct a power-plant to utilize the same flow as at Schaghticoke. The head to be obtained varied from 24 to 39 feet, depending on the amount of water stored behind the dam. Wheels of relatively larger capacity at the maximum head than those at Schaghticoke were thus required in order to get power at low heads when the reservoir was drawn down. As the mean head at Johnsonville would be 28 feet, or about one-quarter of that at Schaghticoke, it was decided to install 5,000 HP. in water-wheels. The average yearly output from Johnsonville was estimated at 13,120,000 kilowatt-hours. The reservoir covers 850 acres and will supply 220 cubic feet per second continuously for 20 days. If filled twice a year from flood-water that would otherwise be wasted it would add during low water more than 2 million kilowatt-hours to the output of the two plants.

The total output from both plants during an average year, and operating at 50 per cent. load factor, would thus be 67,600,000 kilowatt-hours. If it were practicable to dispose of all the power that the flow of the stream would give up to 25,000 HP. continuously, this output might be increased 50 per cent., though, of course, some allowance must be made from these figures for water lost by leakage and sluicing trash and ice.

General Plan of Developments.

The river at Schaghticoke makes a double curve like a letter S, and the general plan decided upon, therefore, was to carry the stream practically in a straight line across both necks of land, thus cutting off about 2½ miles of river with 1 mile of waterway (Fig. 1, Plate 7). This required the construction of a diagonal overflow-dam diverting the water through head-gates into a canal across the first bend of the river.

The low ground and the river crossing made a penstock necessary from the end of the canal to the lower bend. The head of the pipe-line made a natural place for screens, and the high ground of the second neck formed a suitable position for a surge-tank above the power-house. From this tank smaller penstocks could then be taken to the water-wheels.

At Johnsonville a scheme comprising an overflow dam with sluice-gates to control the flow, and a power-house at one end of the dam, was adopted.

SCHAGHTICOKE WORKS.

Dam, Canal and Pipe-lines.—Both a railway and a highway pass across the pond made by the dam, and the height of the Schaghticoke dam was therefore limited by this railway-bridge and also by another power-plant farther up stream. The railway-bridge and the approaches to it were raised 2 feet so as to give 7 feet clearance over the crest of the dam, and the slopes of the embankment were rip-rapped with stone facing; the highway was also raised above flood-water. Large culverts were built to pass the water under the roads in case the pond was drawn down suddenly.

The dam was built diagonally across the river so as to lengthen the spillway and to prevent excessive elevation of the backwater. This also shortened the canal, which would otherwise have had to be extended with a 70-foot cut or tunnel. Good rock foundations existed and the dam followed in general that part of the river-bed where the bottom was highest. Concrete was the material used for the dam, this being made of local gravel (which contained an excess of sand), mixed with broken limestone. About 10 per cent. of large stone was bedded in the concrete.

The length of the spillway is 680 feet, and it is designed to pass a flow of 20,000 cubic feet per second with a height over the crest of 3·9 feet, or to pass 28,000 cubic feet per second with a height of 5 feet. In addition, the sluice-gates in the dam will discharge 3,500 cubic feet per second.

Three gates, 6 feet wide and 9 feet high, are provided (Fig. 1, Plate 7), one at the up-stream end to supply water to the mills on that side of the river and two at the lower end, through which the stream flowed during construction.

Brass-pipe sockets are provided in the crest into which 1-inch pins are inserted to hold flash-boards, which increase the storage by 3 feet. Ordinarily the flash-boards are removed before high water occurs, but in case of sudden flood the pins yield as soon as the water flows over the boards, which are thus released. The pins and part of the flash-boards were in one case carried away by the high water.

The canal intake has eight gates, 6 feet wide and 9 feet high, duplicates of those in the dam, and the maximum velocity of flow through the gate openings is less than 4 feet per second. A curtain-wall 11 feet deep and extending 4 feet below the crest of the dam, protects the gates from injury and diverts ice and logs from the canal to an ice-shoot in the dam.

The canal is 2,300 feet long, and the bottom is rock throughout its entire length, while the sides are partly rock and partly clay and sand. The width at bottom is 20 feet and the side slopes are 1 in ¼ in rock and 1 in 2 in earth. Part of the slopes are in quicksand and are protected by timber cribs under water.

The maximum velocity of flow in rock is 4 feet per second, and in earth 2 feet, with a flow of 1,500 cubic feet per second. The loss of head in the canal varies from 0·4 to 0·8 foot, depending on the depth of water.

The ground at the lower end of the canal is below high-water level, and the material excavated, therefore, was used to build dikes on each side of the penstock intake.

The pipe intake is provided with a floating ice-boom and racks, through which the water passes with a velocity below 2 feet per second at low water. There is an ice-shoot at the side of the intake with a 2-foot sluice-gate below it to drain the canal.

From the intake a penstock, 12½ feet in diameter and 850 feet long, runs over the river and across the lower neck to the surge-tank. This pipe is of riveted steel ⅜-inch thick, with stiffening rings of steel angle-bar every 12 feet. It is supported by reinforced-concrete piers every 12 feet, except on the bridge where steel cradles are employed; the bridge consists of two heavy deck-trusses of 92½ feet span each. At the river-crossing the pipe-line drops down some 40 feet, rising again to pass through a 70-foot cut above the power-house; an expansion-joint is provided at each end with an 18-inch drain-valve at the lowest point, and a footway was built along the top of the pipes for convenient access to the power-house. The maximum velocity in the pipe is 12·2 feet per second, and the loss of head with 1,500 cubic feet per second flowing is about 2 feet.

The surge-tank is 40 feet in diameter and 56 feet high, being made of steel $\frac{9}{16}$ inch thick at the bottom and ⅜ inch at the top. It has sufficient capacity to permit of full load being instantly thrown off or on the water-wheels, without the water either overflowing or being drained below the level of the outlet pipes.

From the surge-tank four 6-foot pipes, 320 feet long and made of ⅜-inch riveted plate, lead to the main water-wheels, and a 2-foot pipe is led to the exciter wheels. These pipes are buried to protect them from freezing.

Gates with by-pass valves and air-relief pipes are built in the surge-tank at the head of each 6-foot pipe, being operated by electric motors controlled from the power-house. Further, so that the change in flow may be gradual and little velocity head may be lost, bell-mouths are placed inside the surge-tank at each outlet.

The velocity with 375 cubic feet per second flowing in each 6-foot pipe is 13·3 feet per second, and the loss of head then amounts to about 2½ feet.

Power-House.—A cross-section of the power-house is shown in Fig. 2, Plate 7. It is a steel-frame brick building, 152 feet long by 52 feet 10 inches wide. The generator and turbine room measures 32 feet by 131 feet 10 inches, and is covered by a 20-ton electric travelling crane. At the back of the turbine-room are compartments occupied by transformers, switches and auxiliary apparatus, distributed over three stories, and a space at the end of the building is devoted to offices, store-rooms, lockers, baths and steam-heating plant.

In order to transport material and machinery to the power-house, the seven-span, concrete-arched bridge, 250 feet long, shown in Fig. 1, Plate 7, was built across the river near by.

Four 5,000-HP. hydraulic turbines with vertical shafts and two 250-HP. exciter-wheels with horizontal shafts are installed. These work under heads varying from 140 to 155 feet, depending on the water-levels in the pond above the dam, on the river below and on the load on the plant.

The turbines (Fig. 2, Plate 7) are of the Francis, radial inward-flow type. The main units run at a speed of 300 revolutions per minute and the exciter-wheels at 600 revolutions. All have cast-iron scroll-cases, bronze runners and pivoted vane-gates. The main wheels are 52 inches in diameter, and the scroll-case tapers down from 5 feet in diameter.

There are two steady-bearings, one above the water-wheel and the other above the generator-field, and the weight of the revolving parts, amounting to 40,000 lbs., is carried by a roller thrust-bearing at the top. The bearings are lubricated by a small pump on each unit, which takes oil from the well below the lower steady-bearing and delivers it to the tank surrounding the thrust-bearing.

The efficiencies obtained from tests of two wheels were as follows:—

Brake HP.	Unit No. 2.	Unit No. 3.	Brake HP.	Unit No. 2.	Unit No. 3.
	Per Cent.	Per Cent.		Per Cent.	Per Cent.
1,000	62·4	..	3,500	83·4	85·0
1,500	69·8	..	4,000	85·0	85·6
2,000	74·7	..	4,500	84·5	85·0
2,500	78·5	81·1	5,000	82·5	83·1
3,000	81·3	83·1	5,400	80·0	..

These tests were made while the plant was in regular operation, the speed being maintained at 300 revolutions per minute, but the effective head varied from 143 to 150 feet.

The governors, mounted on the side of the casting which supports the alternator, are operated by oil supplied under a pressure of 150 lbs. per square inch from duplicate motor-pumps, and the governor-piston opens and closes the vanes of the wheel-gate by means of a ring actuated through a rocker-shaft and cranks. For parallel operation there is a 2-per-cent. drop in speed between full load and no load, and the speed can be adjusted either by hand at the turbine, or from the switchboard by means of a small electric motor on the governor, and a brake is provided to hold the rotor stationary and to stop it quickly, and the electric controller to open and close the gate at the head of the pipe-line is alongside the governor. The fly-wheel effect of each unit is 435,000, or 40,400 lbs. at 3·28 feet radius, and the speed regulation obtained from test was as follows:—

Full load off or on	19	per cent.	variation.
¾ ” ” ”	12	”	”
½ ” ” ”	6	”	”
¼ ” ” ”	3	”	”

The tests were made with one of the 5,000-HP. units loaded on a water rheostat, which was erected just outside the power-house, the governor operating in from 1½ to 2 seconds. The maximum rise in pressure with load thrown off was about 60 lbs. per square inch, or twice normal, and at this pressure the penstocks have a factor of safety of 4 against bursting. Under normal conditions of load the regulation is better than the foregoing figures suggest, as the motors and rotary-converters on the system act somewhat as fly-wheels, and assist in keeping the speed of the generators more constant with variations of load. For instance, with two of the units loaded to nearly full capacity and running the railway at Schenectady, the speed-changes with variations in load of 1,000 or 1,500 kilowatts is only from 1 to 1¼ per cent.

One 3,000-kilowatt alternator is mounted on each of the four water-wheels and supplies three-phase current at 4,400 volts, and 40 cycles per second; the alternators have an efficiency of 96·2 per cent. at full load, including friction, windage and field excitation.

Two 150-kilowatt continuous-current generators supply power at 250 volts for field excitation and for such station auxiliaries as gates, pump- and crane-motors, oil-switches, lighting, etc. Each exciter is driven by means of a water-wheel and is of sufficient capacity to excite all four alternators and to supply all of the auxiliaries.

Four three-phase, 3,000-kilowatt transformers are installed, raising the line-pressure to 32,000 volts. Each is contained in an oil-tank enclosed in a brick compartment with steel rolling doors, and they are cooled by water from the exciter penstock circulating through coils of pipe submerged in the oil.

All switches are of the oil type, electrically operated from the switchboard either by motors or solenoids. The control switchboard is situated centrally in the gallery, and the connections are indicated by dummy bus-bars. The low-tension switches, bus-bars, etc., below the gallery and the high-tension above it are all enclosed in brick compartments.

THE JOHNSONVILLE WORKS.

A cross-section of the Johnsonville power-house is given in Fig. 3, Plate 7, and the general plan of the works is shown in Fig. 4, Plate 7.

The design of the overflow dam is the same as that at Schaghticoke, the maximum height being 40 feet with a length across the spillway section of 530 feet. Four sluice-gates, operated either by hand or electric motors, are placed at the north end; they measure 6 feet by 9 feet and have a combined capacity when the reservoir is full of 8,000 cubic feet per second. Grooves for stop-logs are placed in front of the gates, so that the openings may be shut off for the gates to be inspected or repaired. Next to the sluice-gates is an ice- and log-shoot, with its crest 3 feet below the spillway of the dam, and the opening, when not required, is closed by two sets of stop-logs. A deflecting wall, with three submerged openings extends diagonally from the ice-shoot to the concrete retaining wall which supports the clay hardpan bank. This deflecting wall is 2 feet thick and is reinforced in both vertical faces, as it will be subjected to ice-thrust on both sides; two reinforced-concrete beams further tie it back to the retaining wall.

At the south end of the dam there is an abutment from which a core-wall about 180 feet long extends back to high ground to prevent seepage around the end of the dam; this wall is carried down to bed rock and has a maximum height of about 25 feet, being covered by an earth embankment. The abutment runs into a retaining wall which curves towards the river below the dam in order to divert the water away from a mill-building about 120 feet lower down.

On the north side again, immediately in front of the forebay, are two wheel-flumes separated from it by the gates and screens. Each flume is 20½ feet wide in the clear, 27½ feet long and 35 feet high

(Fig. 3, Plate 7). The walls of reinforced concrete 4 feet thick are designed to withstand the thrust of the water with both flumes full, or one empty and the other full. The reinforcement is carried back to the retaining wall on one side and to the dam on the other, which gives the necessary weight for stability. A reinforced concrete curtain-wall, 3 feet thick and extending down 17 feet from the top of the flumes, was constructed across the intake, and the floor laid over the flumes forms a platform extending back to the power-house. At the back of the two intake openings are the flume-gates, and behind these are the inclined racks. The flume-gates are of structural steel, 18 feet 9 inches high and 21 feet 2 inches wide. In addition to the angle-iron seats set in the concrete, there are in each opening two vertical beams which extend from the floor of the flume upwards to a girder of reinforced concrete, 3½ feet high and 5 feet 2 inches deep, forming the bottom of the curtain-wall. Each gate weighs about 7 tons and is raised and lowered by two steel screws connected to a pair of hoisting stands, which are mounted at the top of the flume and worked by either hand-wheel or motor. Before raising a gate the flumes may be filled and the water-pressure balanced by means of 24-inch by-pass valves.

In each flume is mounted a pair of 57-inch horizontal water-wheels of the Francis inward-flow type, discharging into a common draft-chest constructed between the runners. These runners have wicket-gates which are opened and closed by a common rotating-shaft. The wheels will develop 3,000 HP. at 35 feet head, and are designed to operate satisfactorily when the reservoir is drawn down to a head of 24 feet, in which case the output will be reduced to 1,750 HP., the speed remaining constant. The shafts enter the power-house through a cast-iron, watertight bulkhead ring concreted in the wall. Each wheel is equipped with a Lombard oil-pressure governor.

The turbines are guaranteed to have an efficiency of 82·5 per cent. when working at ¾ gate and 35 feet head; at 24 feet head the efficiency is 78 per cent. at full gate, and 71·5 per cent. at 0·8 gate. As yet no efficiency tests have been made on this apparatus.

Each of the 3,000-HP. wheels is direct coupled to a 1,800-kilowatt alternator, running at 150 revolutions per minute and supplying three-phase current at 4,400 volts, 40 cycles. There is on the same shaft a 60-kilowatt exciter of sufficient capacity for both generators. The alternators have an efficiency of 96 per cent. at full load.

Two 1,800-kilowatt water-cooled oil-transformers raise the generator voltage from 4,400 to the potential of 32,000 volts for the transmission line. The transformers and switchboards are installed at the north end of the power-house, the former being in separate fire-proof compartments, with steel rolling doors over the openings.

The switchboard panels are on a gallery between the transformer compartments, and above this on a floor extending across the building are the generator and line oil-switches, controlled by motors; also the instrument transformers, lightning arresters, etc. Each 1,800-kilowatt transformer when tested showed a full-load efficiency of 98·2 per cent.

The power-house building (Fig. 3, Plate 7) is of reinforced concrete and steel, the inside dimensions being 80 feet long, 31 feet 4 inches wide, and 40 feet 6 inches high. The operating floor is 67 feet long.

The walls separating the wheel-flumes from the generating-room are 4 feet thick, and a 1 : 2 : 4 wet gravel concrete was used for these walls, without the broken stone used in the other cases. This gives a dense masonry that prevents seepage from the flumes into the power-house, and the generating-room wall is absolutely dry without any waterproofing under a water-pressure of 30 feet.

There are two tail-race openings for each pair of water-wheels, the arched walls of which support the generating-room floor. From the front of the power-house, and forming the banks of the tail-race are two concrete walls, one a retaining wall, having a maximum height of 24 feet and a length of about 100 feet, which supports the road to the power-house, and the other a diverting wall curved toward the centre of the stream and about 120 feet long, to divert the water from the ice-shoot and sluice-gates away from the tail-race.

TRANSMISSION LINES.

Three three-phase 32,000-volt circuits pass out of the Schaghticoke power-house, two lines running to Schenectady and the third being the feeder from the Johnsonville plant, which is connected to the high-tension busbars in the Schaghticoke generating-station. There is also a 4,400-volt circuit leaving the power-house, to which the mills at Schaghticoke (formerly driven by their own low-head water-powers) are connected.

The high-tension circuits pass through porcelain bushings in the roof (Figs. 2 and 3, Plate 7), and alongside of these are horn-gaps in series with aluminium-cell electrolytic lightning arresters.

The line to Schenectady is 21 miles long and is supported by galvanized steel towers (Fig. 5, Plate 7), with standard spans of 600 feet. It consists of six stranded copper cables, No. 000 B. and S. gauge (0·409 inch), a $\frac{3}{8}$-inch diameter stranded steel cable, and two No. 6 B.W.G. galvanized-iron telephone wires. The steel cable is attached to the top of the tower 5 feet above the upper cross-arm as a protection to the line from lightning disturbances. There are three cross-arms, about 17 feet wide, from the ends of which the copper conductors are carried, one three-phase circuit being on each side of the tower. The height from the ground to the lower cross-arm of the standard tower is 51 feet, and the distance between cross-arms is 10 feet. The total height of the tower above the ground is 76 feet, and it is 17 feet square at the base with legs set 6 feet in the ground; its weight is about 3,500 lbs. This tower will stand a horizontal pull in any direction of 9,000 lbs., distributed over the cross-arms, and was designed for a wind-velocity of 40 miles per hour with ½-inch coating of sleet on the wires and a temperature of 40° F. below freezing. For angles in the line, a specially heavy tower is used, with the legs set in concrete; this tower is of the same general design as the standard, except that it is 20 feet square at the base, is about three times as strong, and weighs 11,600 lbs.

Two complete spirals are made in the line, six transposition towers (Fig. 6, Plate 7), equally spaced along the line, being used for this purpose. In making a transposition the upper phase is brought to the bottom, the middle phase to the top, and the bottom phase to the centre arm; each transposition tower thus giving one-third of a revolution.

The line crosses the Hoosic River at the power-house, and various other rivers, canals and railways en route. The span over the Hoosic River at the power-house is anchored by six steel poles set in concrete on the hillside behind the power-house. Two single-circuit towers on the opposite bank of the stream support the conductors in a horizontal plane for convenient connections to the outlets in the roof of the building. The Hudson River is crossed by one 1,125-foot span, and the Mohawk River near Schenectady by a number of spans, the longest being 1,263 feet on towers 96 feet high, under which the proposed New York State barge-canal will pass.

All of the towers were shipped knocked-down, and they were then erected in the field. Where the soil is soft or swampy, and at all angles and special crossings, the legs of the towers are set in concrete foundations.

The copper conductors and the lightning guard-wire both have a tensile strength of 7,500 lbs. The former was pulled up at a tension of about 2,500 lbs., corresponding with 10 feet sag.

The telephone line is anchored and transposed at every tower, and is strung from the lower cross-arm with a greater sag than the copper conductors. It is connected to telephones every 2 miles, with telephone transformers in circuit at each station, so that the patrolmen may use the instrument without risk of electric shock.

The insulators consist of porcelain disks of the General Electric suspension and strain types, the latter being used on all angle towers and wherever it was necessary to dead-end the lines. After completion the line was tested by applying a potential of 60,000 volts for several hours.

The Schaghticoke line has a capacity for delivering 25,000 HP. in Schenectady with 7½ per cent. loss. With one circuit cut out by an accident or for repairs the full capacity may be delivered over the other, though at twice this loss.

At Johnsonville the 32,000-volt line leaves the power-house through the north wall of the building, the lightning arresters being inside the power-house and the horn-gaps on the roof as at Schaghticoke. Provision is also made for carrying a 4,400-volt circuit through the wall for a local feeder at Johnsonville. The high-tension line is 5·8 miles long and of much lighter construction than the Schaghticoke-Schenectady line. It consists of galvanized steel towers supporting a single circuit, consisting of three solid copper wires of No. 2 B. & S. gauge (0·257 inch diameter), a No. 2 B.W.G. galvanized iron guard-wire, and two No. 6 B.W.G. galvanized-iron telephone-wires. The tower will stand a side-strain of 3,000 lbs. at the cross-arm and weighs 1,500 lbs.; it is 16 feet square at the base and the cross-arm is 46 feet 9 inches from the ground, and the standard spans are from 500 feet to 600 feet. The conductors are 5 feet 6 inches apart with the steel guard-wire 2 feet 9 inches above the cross-arm. The wire was pulled up to a tension of 600 lbs., corresponding with about 18 feet sag.

The sub-station at Schenectady is situated at the works of the General Electric Company, where 15,000 kilowatts of transformers are installed to step down to 10,000 volts for distribution.

The Schaghticoke work was started in August, 1907, and power was delivered in Schenectady on the 1st December, 1909. The construction of the Johnsonville plant was begun in June, 1908, and completed in June, 1909.

The scheme was designed and carried out by the firm to which the Author belongs, Messrs. Viele, Blackwell and Buck, of New York.

The Paper is accompanied by nine black-and-white prints and numerous photographs, from which Plate 7 has been prepared.

OBITUARY.

JAMES BINGHAM ALLIOTT, born at Nottingham on the 28th November, 1844, died at Brighton on the 31st March, 1910. The son of Mr. Alexander Alliott, a founder of the business of Manlove, Alliott and Company, Limited, manufacturing engineers, of Nottingham, he entered the firm in his youth, was subsequently taken into partnership, and at the time of his death, was Chairman of the company. His name was associated with many improvements in sugar-making processes and in other machinery. Mr. Alliott was elected a Member of The Institution on the 20th May, 1890.

FRED SMITH BUTTON, born at Barnsley in 1859, died suddenly at Manchester on the 21st March, 1910. Trained in the Borough Engineer's Office of his native town, he became Borough Engineer and Surveyor at Accrington and later at Burnley, resigning in 1895 to engage in private practice. He was well known in connection with sewerage matters and acted as Consulting Engineer to many local authorities. He was elected an Associate Member of The Institution on the 4th December, 1888, and was transferred to the class of Members on the 8th January, 1895.

ALFRED COLSON, born at Newport, Monmouth, on the 31st January, 1849, died on the 27th May, 1910, at Leicester, where he had been Gas Engineer to the Corporation for 28 years. Appointed in 1882, after some years' similar experience in Birmingham, he carried out many improvements and extensions, and later successfully undertook also the management of the electrical department A well-known figure in gas-engineering circles, he was the patentee of many inventions, especially for dealing with residuals, and frequently acted in a consultative capacity. He was elected an Associate Member of The Institution on the 7th March, 1886, and was transferred to the class of Members on the 9th April, 1889.

CHARLES HENRY DENHAM, eldest son of the late Admiral Sir Henry M. Denham, was born at Parkfield, Liverpool, on the 13th April, 1833, and died at Southsea on the 18th April, 1910. Joining the staff of the East Indian Railway in 1852, he served 37 years with that Company, occupying for several years the position of Chief Engineer. He was a joint inventor of the Denham-Olpherts cast-iron sleeper and chair, which has been extensively employed in India. Mr. Denham was elected a Member of The Institution on the 4th February, 1868.

ARTHUR GREENWOOD, born in October, 1845, died at Burley-in-Wharfedale on the 19th January, 1910. After being privately educated at home and on the Continent, he entered, as an articled pupil, the firm of Greenwood and Batley, of Leeds, of which his father, Mr. Thomas Greenwood, was one of the original partners. After acting as one of the shops' foremen, he became under-manager, and at the age of 25 was made a partner in the firm. He was elected Chairman on its conversion to limited liability in 1888. The great development of the firm, which manufactured turbines, torpedoes, small arms and other machinery for Government and private users, was due largely to Mr. Greenwood's good management and business acumen. He was a Justice of the Peace for the city of Leeds, and took great interest in local affairs, especially in the University of Leeds. Latterly he held the office of Chairman of the Engineering Employers' Federation. He was also a member of council of the Institution of Mechanical Engineers.

Mr. Greenwood was elected a Member of The Institution on the 6th December, 1881.

CHARLES ALFRED HASBROUCK, born at Ithaca, New York, on the 31st July, 1864, died at Pasadena, California, on the 1st February, 1910. Graduating from Cornell University in 1884, he joined the staff of the Detroit Bridge and Iron Company, serving subsequently with the King Bridge Company and the American Bridge Company of New York, of which he became Contracting Manager in 1904. His career as a bridge and structural engineer extended over a period of 24 years. He was elected a Member of The Institution on the 2nd February, 1904.

CHARLES WILLIAM HODSON, C.S.I., born on the 6th September, 1851, died at Ealing on the 15th February, 1910. Obtaining his scientific education at Coopers Hill College and his practical training under Messrs. John Aird and Sons, he entered the Public Works Department of India in 1873. Until 1882, when he was promoted to the rank of Executive Engineer, he was engaged on the Rajputana, Tirhoot and Indus Valley State Railways. Subsequently he was employed on the Hurnai and Quetta divisions of the Sind-Peshin (State) Railway, as Superintendent of Way and Works on the Frontier section of the North-Western Railway, and as Engineer-in-Chief of the Mushkaf-Bolan Railway. In 1896 he became Consulting Engineer and Joint Secretary to the Madras Government, and 2 years later he was appointed Director of Railway Construction to the Government of India, retaining this post until his retirement in 1906. He received the Companionship of the Star of India in 1905.

Of outstanding ability and untiring energy, Mr. Hodson was also gifted with the faculty of inspiring all who worked with him with the same devotion to duty which he himself possessed. With a fine presence and a genial kindly manner, he was a favourite with all who came in contact with him officially or otherwise. His all too early death was sincerely regretted by the very many officers of the Indian Public Works Department who had known and worked with him in the great days of Frontier railway construction in India.

Mr. Hodson was elected an Associate Member of The Institution on the 4th February, 1879, and was transferred to the class of Members on the 5th November, 1891. From 1904 to 1906 he served as a representative of India on the Council.

HENRY ALBERT HOY, born on the 13th January, 1855, in London, died at Manchester on the 24th May, 1910. Trained on the London and North Western Railway under the late F. W. Webb, he served 20 years, from 1884, with the Lancashire and

Yorkshire Railway Company, becoming Manager of the Locomotive Works first at Miles Platting and then at Horwich, where the electrical and other machinery required on the system was made, largely to Mr. Hoy's design. Later he was appointed Chief Mechanical Engineer, in which capacity he had charge, under Mr. J. A. F. Aspinall, General Manager, of the electrification of the Liverpool and Southport line and many other important works. In 1904, at the invitation of the Directors, he undertook the reorganization and management of Messrs. Beyer, Peacock and Company's locomotive works. Mr. Hay was elected an Associate Member of The Institution on the 1st December, 1891, and was transferred to the class of Members on the 16th February, 1907.

ARTHUR JOHN HUGHES, C.I.E., born in 1843, died at Eastbourne on the 9th March, 1910. Trained under his father, the late Mr. John Hughes, he went to India in 1862, and during a career of 34 years in the Public Works Department, he filled important administrative offices, and will be especially remembered in connection with the great engineering works for the drainage and water-supply of Calcutta, Allahabad, Cawnpore and other cities.

Mr. Hughes was elected a Member of The Institution on the 6th December, 1881.

HENRY JOLL, born at Stoke, Devonport, on the 4th January, 1845, died at Plymouth on the 15th April, 1910. He joined the Public Works service of India in 1867, and after a varied experience of 25 years on roads, bridges, canals and other works, he retired with the rank of superintending engineer in March, 1892, on account of failing health. Mr. Joll was elected an Associate of The Institution on the 1st March, 1870, and was transferred to the class of Members on the 27th March, 1877.

JAMES CHATBURN MADELEY, born at Moss-side, near Manchester, on the 17th November, 1840, died at Guildford on the 29th January, 1910. He served his pupilage to the late Sir James Brunlees, Past-President, who later sent him out to Brazil as an assistant engineer, under Mr. D. M. Fox, on the construction of the São Paulo Railway, where he was engaged on the famous inclined planes at the Serra do Mar, and remained until the opening of the line in February, 1867. He was then employed as Engineer on the survey and construction of a projected interoceanic railway in Spanish Honduras, subsequently abandoned for lack of funds, and Engineer and Superintendent of the Portalegre and New Hamburg Railway, Brazil. Resigning this appointment in 1885, he returned to England and afterwards travelled abroad. After fulfilling an engagement as Engineer to the Lake Copais Drainage Company, he in 1892 rejoined the staff of the São Paulo Railway as Engineer and Locomotive Superintendent to the Company, carrying out many important improvements and extensions. He retired in 1897, and spent his remaining years in England. Mr. Madeley was an engineer of considerable experience and ability, very painstaking in all he undertook, thoroughly reliable and high principled, and devoted to his profession. He was elected an Associate of The Institution on the 6th December, 1870, and was transferred to the class of Members on the 19th February, 1878.

WILLIAM METCALF was born on the 3rd September, 1838, at Pittsburgh, Pennsylvania, where he died on the 5th December, 1909. He was engaged all his life in the manufacture of iron and steel, formerly with the Fort Pitt Foundry and the Crescent Steel Company, and for the past 12 years as President of the Braeburn Steel Company. He was a Past-President of the American Society of Civil Engineers. Mr. Metcalf was elected a Member of The Institution on the 3rd March, 1896.

EVELYN FRANCIS SANDERS, B.A.I. (*Dubl.*), born on the 12th July, 1864, died at West Hill, Titchfield, on the 31st December, 1909. In 1887 he went out to India as assistant engineer on the construction of the Bengal-Nagpur Railway, becoming successively District, and Superintending Engineer, and in 1904, Chief Engineer of the system. Mr. Sanders was elected an Associate Member of The Institution on the 3rd February, 1891, and was transferred to the class of Members on the 31st December, 1909.

JOHN FRANKLYN TYLER, born at Potton, Bedfordshire, in 1841, died at Hampstead on the 22nd February, 1910. A pupil and assistant of the late Mr. B. S. Brundell, he was employed under that gentleman, between 1869 and 1882, on the Don drainage works, sewerage, sewage-disposal and water-supply works for Doncaster and sewage-disposal works for Wigan. He was last engaged, under Messrs. Holme and King, on the laying of a section of the pipe-line between Lake Vyrnwy and Liverpool.

Mr. Tyler was elected a Member of The Institution on the 6th December, 1881.

GEORGE WHALE, born in 1842, died at Hove on the 7th March, 1910. After being educated at a private school at Lewisham, he proceeded, in 1858, to Wolverton as a pupil to the late Mr. J. E McConnell, who was then in charge of the southern division of the London and North-Western Railway, and 7 years later he was transferred to Crewe, where he worked under Mr. John Ramsbottom for 2 years.

In 1867 he was appointed assistant to the late Mr. J. Rigg, who was then superintendent of the locomotive running department; and on Mr. Rigg's retirement, in 1877, he was made superintendent of the locomotive running department for the northern division, the late Mr. J. Mumford having charge of the southern division. In 1899 Mr. Mumford retired, and Mr. Whale was made superintendent of the running department for the whole line. On the retirement of the late Mr. F. W. Webb, in 1903, Mr. Whale was appointed locomotive superintendent, a position he continued to hold until last year, when ill-health compelled him to relinquish this work. During his tenure of office as Chief Mechanical Engineer on the London and North-Western Railway, the locomotive stock underwent a very considerable change, and the once familiar three-cylinder compound-engines were nearly all withdrawn from service or converted.

Mr. Whale was elected an Associate Member of The Institution on the 6th April, 1886, and was transferred to the class of Members on the 22nd December, 1903.

HENRY JOHN INWOOD BILTON, born on the 21st March, 1862, at Christchurch, N.Z., died at Brighton Beach, Victoria, on the 23rd March, 1910. After serving his pupilage with Messrs.

John Anderson and Sons, of Christchurch, he went to Victoria, where in 1886, he entered the service of the Victorian Railways, in which he was employed as an Assistant Engineer, specializing in hydraulic work, until his death. He was elected an Associate Member of The Institution on the 1st March, 1892.

HENRY LOWTHIAN CLEAVER, born on the 27th August, 1862, died at Calcutta on the 15th December, 1908. Passing out of Coopers Hill College, he entered the service of the Public Works Department of India in 1886, and was employed for some years on coal-fields and railways in Wardah State. From 1898 until his death, he was engaged on the Eastern Bengal State Railway, latterly with the rank of Executive Engineer. Mr. Cleaver was elected an Associate Member of The Institution on the 4th December, 1888.

BERNARD GODFREY, born on the 19th April, 1870, died on the 15th April, 1910, at Brantford, Ontario, where he was formerly engaged on the extension of the waterworks. Trained in the sanitary branch of the profession, he had previously occupied the posts of Borough Surveyor of Droitwich, and Surveyor to the Rotherham Rural Sanitary Authority. He was at one time Honorary Secretary of the Birmingham Association of Students. He was elected an Associate Member of The Institution on the 21st May, 1895.

ARTHUR WERNER ITTER, born on the 14th May, 1854, died on the 25th February, 1910. For the past 17 years he was exclusively engaged in the manufacture of pressed common bricks known as "Flettons," owning extensive modern works at Whittlesea, Cambridgeshire, and at Calvert in Buckinghamshire. At the time of his death he was one of the largest individual brick-masters in the country. Mr. Itter was elected an Associate Member of The Institution on the 2nd March, 1880.

HARRY JAMES MOLLOY, born in Ireland in 1865, died in India on the 3rd March, 1909. Joining the Public Works Department in 1890, he was employed until his death on railways, bridges, roads, buildings, water-supply and other works in various districts of India. In 1903 he was promoted to the rank of officiating Executive Engineer. Mr. Molloy was elected an Associate Member of The Institution on the 3rd December, 1895.

JOHN PARKER, born on the 4th May, 1849, died at Darjeeling on the 19th February, 1910. Trained under his father, who was for many years Surveyor to the Poplar Board of Works, he spent several years on railway construction in New Zealand, and was appointed in 1874 Surveyor to the Basford (Notts.) Rural District Council, a position which he held for 20 years, carrying out important drainage and other works. He subsequently devoted himself to the manufacture and sale of the Parker-Hassall joint pipe, of which he was one of the inventors. Mr. Parker was elected an Associate Member of The Institution on the 24th May, 1887.

EDWARD PILLOW died at Norwich on the 28th April, 1910, aged 58. Apprenticed to Messrs. Shepherd, Hill and Company, Leeds, he received an appointment at the Crewe Locomotive Works, where he subsequently had charge for 17 years of the brass foundry and held other responsible positions, especially in connection with the educational work of the Mechanics' Institute. In 1891 he was appointed organizing secretary to the Norfolk Technical Education Committee, a position which he retained until a short time before his death. He was elected an Associate Member of The Institution on the 1st March, 1881.

STIRLING SMEATON, born at Norwood, South Australia, on the 30th October, 1860, died at Adelaide on the 12th March, 1909. Graduating in Arts at the University of Adelaide, he served 7 years in the Public Works Department of South Australia. Subsequently he was employed on various railway contracts in South Australia, Victoria, and Western Australia, eventually, after a long period of ill-health, re-entering the South Australian service as assistant-surveyor and draughtsman. Mr. Smeaton was elected an Associate Member of The Institution on the 5th December, 1893.

MAJOR JOHN FREDERICK ADOLPHUS McNAIR, C.M.G., R.A., born at Bath on the 23rd October, 1828, died at Brighton on the 17th May, 1910. Obtaining his commission in 1845, he became in 1856 staff officer in the Straits Settlements. Subsequently he was appointed Comptroller of Indian convicts and Surveyor-General to the Colony, in which capacity he carried out important public works in Singapore. During the disturbances of 1875–1876 he was Special Commissioner to Perak, receiving the medal with clasp, and later he took part in special missions to Siam and officiated for some years as Lieutenant-Governor of Penang. He was elected an Associate of The Institution of the 3rd December, 1872.

SIR FREDERICK THORPE MAPPIN, BART., born in 1821, died at Sheffield on the 19th March, 1910, in his ninetieth year. Entering his father's cutlery business at a very early age, he became one of Sheffield's most prominent cutlers and steel manufacturers, being elected Master Cutler in 1855. He sat for the Hallamshire division in the Liberal interest for 20 years, and he also took an active part in the public life of Sheffield, especially in connection with technical and higher education, as well as in the direction of various important business undertakings.

Sir Frederick Mappin was elected an Associate of The Institution on the 7th February, 1865.

JOHN WADDINGTON died on the 9th July, 1910, at Lancing, Sussex, in his 89th year. Trained in the locomotive department of the Hull and Selby Railway, he afterwards joined the firm of Bray, Waddington and Company, manufacturing engineers, of Leeds, who constructed the ironwork for Victoria Bridge, the first railway bridge over the Thames at London, in 1859. Mr. Waddington had been a very active worker in an engineering business in London since 1866, along with his sons. He was elected an Associate of The Institution on the 2nd December, 1862.

*** The following deaths have also been made known between the 30th April and the 18th October, 1910:—

Members.

Allan, George; *died* 2 October, 1910.

Anderson, William; *died* 5 April, 1910.

Barreto, Antonio Paulo de Mello; *died* 12 September, 1908.

Brereton, Cuthbert Arthur; *died* 11 September, 1910.

Chatterton, George; *died* 5 July, 1910.

Eachus, George Eedes; *died* 14 July, 1910.

Golla, Lucien Alexander; *died* 18 May, 1910.

Guttmann, Oscar; *died* 2 August, 1910.

Harris, Henry Graham; *died* 12 October, 1910.

Haynes, Robert Henry; *died* 20 August, 1910.

Hughes, Thomas William Robert; *died* 13 October, 1910.

Jacob, Augustus Hamilton; *died* 25 June, 1910.

Latham, John Herbert; *died* 12 July, 1910.

Lopes, George; *died* 28 June, 1910.

Maclaren, James Waddell Boyd; *died* 21 September, 1910.

Martin, Edward Pritchard; *died* 25 September, 1910.

Meik, Patrick Walter; *died* 12 July, 1910.

Mellis, John Charles; *died* 23 August, 1910.

Paddon, John Birch; *died* 7 October, 1910.

Rigby, Jason; *died* 9 September, 1910.

Stelfox, James; *died* 11 September, 1910.

Taylor, Joseph Jex; *died* 18 August, 1910.

Trewby, George Careless; *died* 19 July, 1910.

Wilson, George; *died* 15 April, 1910.

Wright, John; *died* 10 September, 1910.

Associate Members.

Beveridge, John;

Blake, Henry Kynaston; *died* 9 July, 1910.

Bradford, Mark Gordon; *died* 17 June, 1910.

Cockburn, Henry; *died* 17 June, 1910.

Funnell, Herbert Nicholas; *died* 6 June, 1910.

Gray, Henry Holmes;

Hiscocks, Alfred Moser; *died* 12 September, 1910.

Jacobs, Paul George; *died* 8 April, 1910.

Louth, George Edward; *died* 12 August, 1910.

McLean, John; *died* 19 October, 1908.

Potts, Benjamin Langford Forster; *died* 8 September, 1910.

Robinson, Francis Bertram; *died* 28 September, 1910.

Sharpe, Robert; *died* 8 May, 1910.

Smith, Bertram Robert; *died* 3 October, 1910.

Thomson, John; *died* 17 April, 1910.

Tilley, Albert; *died* 15 September, 1910.

Tuckwell, John Morrison; *died* September, 1909.

Associate.

Hart, Charles Frederick, *died* 11 September, 1910.

SECT. III.

ABSTRACTS OF PAPERS IN SCIENTIFIC TRANSACTIONS AND PERIODICALS.

Road-Rollers Driven by Internal-Combustion Engines. BRET.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xliv. pp. 94-103.)

The improvement of the internal-combustion engine has led to its introduction for the propulsion of road-rollers. The steam-roller suffers from serious drawbacks; it requires a constant supply of suitable water, it does not lend itself to even an approximately uniform distribution of load, and, owing to the size of boiler, difficulty is experienced in arranging for a sufficient overlap between the tracks made by the leading roller and those due to the driving-wheels. Two types of roller driven by internal-combustion engines are described, the one weighing 18 tons and the other 6 tons, the former being thus capable of replacing the steam-roller and the latter the horse-roller. As regards the heavier machine, the fuel is petroleum, of which 1·1 gallon is consumed per hour. The load on the forward roller is nearly 4,000 lbs. per foot of face-width, while that on the driving-wheels is about 5,300 lbs., these figures corresponding respectively to 2,900 and 7,000 lbs. in the case of a steam-roller. In the lighter machine an ordinary two-cylinder petrol-engine constitutes the driving-power. The load on the forward roller is, in this case, 1,300 lbs. per foot of face-width and on the driving-wheels practically double. A Table shows the comparative costs incurred in a day's work with the petroleum-driven machine and a steam-roller respectively. The balance is slightly in favour of the latter, though, when credit is taken for the increased output due to the better distribution of the load, the difference almost vanishes. Where the foundation is soft or the road very narrow neither a steam-roller nor a machine like the heavier of the two described is applicable, and a similar comparison to the above is made between the light petrol-driven machine and a roller drawn by two horses, both machines being available for a similar class of work. From this comparison it is shown that two and a half times as much work can be done per day with the mechanically-driven machine as with the horse-drawn roller; thus, although the cost per working-day, sinking-fund, etc., included, is for the former £1 12*s*. 6*d*. and for the latter 19*s*. 3*d*., the work can be done cheaper by the petrol-driven machine, which also enjoys the important advantage of minimizing the time required for the repairs.

I. C. B.

Strengthening of Bridges for Increased Speed of Trains (France and Italy). E. Randich.

(Bulletin of the International Railway Congress, London and Brussels, 1910, vol. xxiv, pp. 1575–1658.)

Particulars of the French ministerial circulars of 1858 and 1877 and of the Regulations of 1891 are given. The first official document in Italy was published in 1888, and the Regulations of 1897, not officially sanctioned, were supplemented and published in 1909. Stress-limits in iron and steel for over- and under-bridges are quoted. It was the Mönchenstein disaster[1] in 1891 which led to a general systematic revision of all existing metal bridges. The conditions adopted were, in France, denoting the stress-limit fixed in the regulations initiating the revision by R, structures showing no defects and not stressed above 1·33 R not being strengthened, structures with calculated stresses between 1·33 R and 1·66 or 2 R not being strengthened till actual tests had been taken in verification, above these limits strengthening or replacement being carried out forthwith (R not to be exceeded in strengthening works). In Italy, the heaviest rolling-loads, or larger if justifiable, were adopted, temperature-variations and centrifugal force being taken into account. In addition to extra plates to main, cross and longitudinal girders, extra lattice-work and strengthened fastenings, the following works have been carried out:—*France.*—Additional main girders, duplicated cross-girders, vertical strengthening of lattice girders, additions to the webs of plate-girders, total replacement, transformation into tubular girders by additional girders, replacement of timber by metal floors. *Sardinia.*—Additional web-members and bracing, strengthening of longitudinals and of fastenings of cross girders. *Italy.*—Additions to lattice-work, strengthening of joints and extra flanges. Frequently, where several bridges of small and medium spans on the same line are to be dealt with, the longest is taken out and replaced by a new bridge, and the old one shortened and used for another span. Main girders are often doubled from another bridge, which is sometimes replaced by a masonry arch.

The weakest parts are always the longitudinals and the bracing under the rails. Bridges of small or medium span are sometimes removed wholesale and replaced during strengthening by a temporary bridge. The results of tests on strengthened bridges are in general good. Iron and steel is used for strengthening in France, and in Italy steel with the admissible stress-limit of iron. Details of cost per pound for various classes of strengthening works, diagrams of shearing- and bending-loads and of type-trains, and Tables of uniform shearing- and bending-loads are given.

W. H. B.

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[1] *See* Minutes of Proceedings Inst. C.E., vol. cvii, p. 456.

Tyngs Island Bridge. J. W. THOMAS.

(Journal of the Association of Engineering Societies, Boston, Mass., 1910, vol. xliv, pp. 273-86.)

This is a suspension foot-bridge connecting Tyngs Island with the west bank of the Merrimac River, a few miles above Lowell. The span between towers is 550 feet, with a 100-foot trestle-approach on the island side and a shorter approach on the mainland side. The main cables are 2$\frac{3}{8}$ inches in diameter and are stressed up to 63·7 tons, with a uniform load of 290 lbs. per lineal foot on the structure. Suspender-rods every 20 feet support the wooden stiffening-truss and roadway, and guy-wires are fixed to prevent undue lateral motion. The effect of temperature on the cables is such as to make a difference of 1 inch in elevation for a 5-degree change in temperature. The bridge is thus subject to a change of elevation of about 2 feet for the 120-degree range in temperature. The towers are of wood 45 feet high, the leg timbers being 10 inches by 10 inches. The anchorages consist of reinforced-concrete slabs 9 feet by 8 feet and 2½ feet thick. The total cost, including approaches, was £1,230.

R. S. B.

Pneumatic-Caisson Bridge-Piers in Alaska.

(Engineering Record, New York, 1910, vol. 61, pp. 559-62.)

A task of exceptional difficulty was the building of the piers for the Copper River and North Western Railway bridge over the Copper River, which is heavily ice-bound for a great part of the year with active glaciers on each side, frequent floating icebergs and a range of 24 feet of water. The timber caissons were carried down 44 feet below low-water with sharp up- and down-stream cutwaters heavily armoured with old rails close together in vertical lines. The piers, 55 feet high, carry the superstructure 30 feet clear of high water, and are solid six-sided masses of 1 : 2 : 4 concrete, 64 feet by 21 feet at bottom and 45 feet by 13 feet at top. The caissons and air-locks, power-house and 1,800-foot cableway, and the many severe climatic difficulties encountered during the progress of the work, are fully described.

C. O. B.

Three-Hinged Reinforced-Concrete Arch-Viaduct.

(Engineering Record, New York, 1910, vol. 61, pp. 576-9.)

This viaduct, 185 feet long, consisting of a 45-foot girder, 81-foot and 45-foot hinged arches, and a 12-foot 6-inch girder, all of reinforced concrete, is remarkable only for the circumstances of erection.

These were that the timber runway-bridge, which the concrete viaduct has replaced on the same site, had to be maintained with its daily 8-hour traffic of live-stock, while the seven-line railway and roads in Chicago, over which the viaduct was built, could suffer no interruption of traffic, nor was there any room between the lines for temporary supports for centering, etc. The means by which the structure was successfully erected, under the above adverse circumstances, are fully detailed.

C. O. B.

Partial Underpinning of the Gorgopotamos Viaduct.

GEORGES BODIN.

(Le Génie Civil, Paris, 1910, vol. lvii, pp. 121-3.)

In the new railway between the Piræus and the Turkish frontier it was intended that the viaduct across the Gorgopotamos should consist of six latticed spans, each 91·84 feet in length, and that all the piers as well as the abutments should be of masonry. During the execution of the work, however, the "bottom" on the right bank of the stream presented such unsatisfactory features that a seventh similar latticed span, which replaced practically the whole of the terminal embankment, and thus relieved the bad ground of a load it seemed ill-fitted to support, was added at that end of the viaduct, while the two adjacent piers were constructed in ironwork. The early completion of the Gorgopotamos viaduct was of paramount importance as many of the more important sections of the railway were dependent for supplies on this route; the foundations of the work on the right bank of the stream, therefore, were not carried down as deep as they would have been if time had not been so important a factor; every care, however, was taken to keep the foundation material well drained, as it was found to be affected most prejudicially by water, and with this object a length of 1,866 feet of gallery surrounding both piers and abutments was constructed.

The works were completed in 1904, but three months after the passage of the first train the abutment showed signs of unequal settlement, eventually accompanied by considerable lateral displacement, the rate of movement increasing, though not uniformly, with time. It was decided, therefore, to underpin with a block of masonry resting on a fairly solid bed, which was struck off at a level of 72 feet below that of the original foundation. The work was carried up from the new bed in lifts of 6·56 feet, each lift being cut up into small areas, so that the support of the abutment over which the traffic continued to circulate should not be dependent to any great extent on the shoring. The work was rendered the more difficult owing to the thrust of the backing, a thrust which had already caused the displacement of the original abutment. The total cube of the underpinning was 3,940 cubic yards, and the cost 48*s*. 6*d*. per cubic yard, including excavation and masonry. The pier, next to the abutment referred to, also showed signs of settlement, but, owing to the absence of lateral thrust and to the smaller difference between the levels of the new and the old foundations, the work of reinstatement was of a simpler nature.

I. C. B.

Crossing of the Seine by the Metropolitan Railway at Île de la Cité. P. CALFAS.

(Le Génie Civil, Paris, 1910, vol. lvii, pp. 41-7.)

The Seine at the point in question being divided into two branches, the works to be executed embraced a double crossing and two low-level stations, while the tunnelling under two existing railways increased the difficulties. The approach-tunnels and the length of work under the Caserne de la Cité were executed by ordinary methods, but for the actual crossings, as well as for the low-level stations, caissons, sunk by the use of compressed-air, were employed. For the crossings, each caisson constituted a length of completed tunnel, three such lengths being required for the larger branch of the river and two for the smaller, while for the stations a short caisson, forming the means of access, was sunk at each end, the intervening space being filled with a continuous length similar to those used for the crossings, but of larger section. The caissons crossing the larger branch of the river couple direct to the station on the Île de la Cité, but between those traversing the smaller arm and the station in the Place St. Michel is a length of 210 feet, over which it was intended that the tunnel should be executed by the freezing process, since it was thought that the strata would be very wet and that the danger to the Orleans Railway, which crosses the site of the new work at this point, would, by the use of that process, be minimized. It was intended to bore, in a direction parallel with the axis of the tunnel, the holes for the pipes used in freezing the ground; thus the permanent work would have been carried out within an encircling tube of frozen ground. The work of drilling these holes was begun from the end caisson of the St. Michel station, but after nearly 100 feet had been penetrated the nature of the ground encountered was so unfavourable that the system was abandoned and the length carried out by ordinary tunnelling, except at the very junction with the caissons forming the crossing, where the freezing process was retained, although with the modification that the pipes through which the cold brine was circulated were placed vertically as usual. Owing to the comparatively favourable nature of the ground, a shield (fully described) was used only for a single length of tunnelling, and even there the use of compressed-air was found unnecessary; thus there was no opportunity of testing the suitability of the design for difficult work.

I. C. B.

New Buffalo Waterworks Tunnel. F. LAVIS.

(Engineering Record, New York, 1910, vol. 61, pp. 802–6.)

The bottom of this concrete-lined tunnel, 10,845 feet long, mostly under Lake Erie, is 65 feet below water with about 20 feet of rock over its top, some of it being limestone of good quality in horizontal strata, but most much harder and full of seams. Two-thirds is 12 feet square and the remainder 9 feet. Nearly all the former was driven under compressed-air, the pressure being 25 to 28 lbs. Details of the methods pursued are given and a comparison is instituted, as regards quantity of explosives used, between this work and the Simplon, Loetschberg and Albula tunnels. Tables show monthly progress, number of employees, with wages, etc. The article is fully illustrated.

C. O. B.

Egyptian State Railways and Telegraphs.

(Report of the General Manager, 1909.)

	1909	1908
	Kilometres.[1]	Kilometres.
4-foot 8½-inch gauge	2,036	2,036
Secondary branches, sidings, etc.	790	790
3-foot 6-inch gauge	222	222
Secondary branches, sidings, etc.	22	22
	£E.	£E.
Gross receipts[2]	3,258,156	3,135,451
Working-expenses	1,974,767	2,082,231
Net receipts . . .	1,283,389	1,353,220
Details of working-expenses—	£E.	£E.
Maintenance	481,848	482,664
Locomotive	652,357	728,727
Carriages and wagons	230,450	215,572
Traffic	401,265	396,977
General	208,847	228,291
Passenger-journeys	25,306,178	25,851,661
Goods-tonnage	5,180,830	5,775,198
Live-stock (head)	34,754	36,243
Train kilometrage	14,963,529	15,821,586
Engine "	19,688,406	20,856,597
	£E.	
Capital	25,293,555	
Rolling-stock—		
Locomotives	585	589
Carriages	1,272	1,274
Wagons	10,891	10,847

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[1] To convert kilometres into English miles, multiply by 0·625.
[2] To convert £E to £ sterling, multiply by 1·015.

TELEGRAPHS.

	£E.	£E.
Earnings	107,789	108,310
Working-expenses	104,048	104,088
Net revenue	3,741	4,222

LIGHT RAILWAY COMPANIES.[1]

Kilometrage	1,228	1,225
	£E.	
Receipts	299,620	
Working-expenses	180,031	
Net revenue	119,589	

C. O. B.

Central South African Railways.

(Report of the General Manager, 31 December, 1909.)

Mileage at end of year—	**1909**	**1908**
3-foot 6-inch gauge single	2,336	Not given.
„ „ double	44	
„ „ treble	7½	
2-foot gauge single	27	
Total	2,414½	
Average mileage worked	2,368	2,332[2]
Mileage under construction	503	Not given.
	£	£
Earnings	5,061,421	4,218,297
Working-expenses	2,232,667	2,420,540
Net receipts	2,831,754	1,797,757
Percentage of working-expenses to gross receipts	44·1	57·4
Train-mileage	7,335,933	6,943,355
Number of passengers	9,152,779	7,742,845
„ live-stock	1,341,401	1,009,649
Goods-tonnage	6,029,522	4,827,961
Goods departmental	534,466	419,629
Receipts per train-mile	165·7*d.*	155·9*d.*
„ „ average open mile	£2,139	£1,809
Working-expenses per train-mile	73·0*d.*	89·8*d.*
„ „ „ average open mile	£942	£1,046
Capital	£25,791,631	£25,282,541

	3-ft. 6-in. Gauge.	2-ft. Gauge.	3-ft. 6-in. Gauge.	2-ft. Gauge.
Number of locomotives	414	2	482	2
„ carriages	543	2	522	2
„ trucks, etc.	7,533	44	7,564	44

Employees—		
Whites	7,738	7,217
Coloured	5,362	4,752

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[1] These are of 3-foot 3⅜-inch and 2-foot 5½-inch gauges.

[2] The figures for 1908 throughout are slightly amended from the previous year's report, explanation being given.

There is a retrospect of the railway-working of 1902–1909, with diagrams, this being the last report of the Central South African Railways as a separate system. A report of the Inspector of White Labour is also included.

C. O. B.

French-Somaliland Railway. GODFERNAUX.

(Revue Générale des Chemins de fer, Paris, 1910, vol. xxxiii, pp. 385-411.)

A concession having been obtained to construct and work for 99 years a line from the port of Jiboutil, on the Gulf of Aden, to the White Nile, via Harar and Adis Abeba, the contract was let at £6,380 per mile, and work began in October 1897. Delays and difficulties were caused by lack of water and by the hostility of the natives, but in 2 years the earthwork was completed up to 62 miles, and track-laying to 32 miles, at which point a viaduct of 450 feet crosses a ravine; further delays occurred, but in December, 1901, railhead reached 127 miles, the line having been open to traffic to 101 miles some time previously. Various financial and international questions then arose, and experts were sent out to report generally on the situation, as well as on the best route to follow from Diré Dawa, which had been reached at the end of 1902, to the present terminus at Adis Abeba. In 1909 the staff charged with continuing the line to this point commenced work, which they are now pushing to completion. The line is of metre gauge with Vignoles rails 32 feet long, weighing 44 lbs. per yard for the first section, and of somewhat heavier type for the other portion. The curves have a minimum radius of 328 feet. The buildings are steel-framed, with brick filling, and roofed with tiles or reinforced concrete. The route, more or less through desert, gradually rises, until at 40 miles an elevation of 2,152 feet is attained; then, after a fall, it rises again at 68 miles to 2,785 feet. At 56 miles the Protectorate is left and Abyssinia is entered. At Diré Dawa (192 miles) the line reaches an elevation of 3,920 feet. The route followed by the extension to Adis Abeba is 300 miles, with its highest points at Assabots, 4,895 feet, and at the terminus which is 7,733 feet above sea-level. The gradients, however, are not heavy, bridges and other works being few, while tunnels are entirely avoided. The locomotives consist of ten Swiss engines of 26 tons, which can haul an 85-ton train at 18 miles an hour on the easier portions of the line, a "Consolidation" engine of 35 tons, and a light shunting or contractors' engine. The article is fully illustrated.

G. H. S.

Cape of Good Hope Government Railways.

(Report of the General Manager, 31 December, 1909.)

	1909	1908
Mileage—		
3-foot 6-inch gauge	3,252	Not given.
2-foot gauge	168	
Total single mileage	3,571	
Sidings	245	
Mileage under construction	107	
	£	£
Earnings	3,129,054	2,850,139
Working-expenses	2,072,341	2,246,881
Net receipts	1,056,713	603,258
Passenger-journeys	15,344,717	16,251,695
Goods-tonnage	1,350,268	1,285,969
Live-stock (head)	1,665,600	1,403,555
Train-mileage	7,292,306	7,160,728
Engine-mileage	9,012,345	9,006,160
	d.	d.
Earnings per train-mile	103·0	95·5
Expenditure per train-mile	68·2	75·3
Detail of working-expenditure per train-mile—		
Maintenance (including new works)	14·3	14·8
Locomotive	28·7	32·3
Traffic	16·7	18·8
General	8·5	9·4
Total	68·2	75·3
	Miles.	Miles.
Average passenger-travel	19	18
„ goods-haulage	182	164
„ passengers per train	72	72
„ goods per train (tons)	93	82
	d.	d.
Receipts per unit-mile passengers	0·892	0·862
„ „ „ goods	1·530	1·672
Expenditure per unit-mile passengers	0·735	0·889
„ „ „ goods	0·837	1·034
Capital expenditure	£32,347,238	Not given.
Rolling-stock—		
3-foot 6-inch gauge—		
Locomotives	658	628
Motor-carriages	4	4
Carriages	1,049	1,037
Trucks, etc.	13,221	12,111
2-foot gauge—		
Locomotives	16	16
Motor-carriages	3	3
Carriages	42	42
Trucks, etc.	305	315

This last report of the Cape Government Railways as a separate system mentions with satisfaction a return towards prosperity as indicated by the figures, and enlarges on the progress of the Government railway-sleeper plantations, of which three photo-reproductions are given. C. O. B.

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Natal Government Railways.

(Report of the General Manager for the year 1909.)

	1909	**1908**
Mileage on the 31st December—		
3-foot 6-inch gauge	1,064	1,064
2-foot gauge	29	..
Under construction	47	Not given.
	£	£
Earnings	2,024,657	1,832,862
Working-expenses	1,187,038	1,240,319
Net revenue	837,619	592,543
Percentage of working-expenses to gross receipts	58·63	67·67
Number of passengers	2,644,612	2,675,426
Goods-tonnage[1]	3,004,744	2,676,748
Live-stock number	206,813	192,495
Train-mileage	5,034,205	4,844,858
Engine-mileage	6,685,387	6,413,550
	d.	*d.*
Earnings per train-mile	96·52	90·79
Working-expenses per train-mile	56·59	61·44
Net return per train-mile	39·93	29·35
	£	£
Earnings per average open mile	2,051	1,877
Working-expenses per ditto	1,202	1,270
Net return per open mile	849	607

	Per Open Mile.	Per Train-Mile.	Per Open Mile.	Per Train-Mile.
Detail of working-expenses—	£	*d.*	£	*d.*
Maintenance	166	7·81	195	9·43
Locomotive	648	30·50	669	32·36
Traffic	316	14·86	343	16·58
General	72	3·42	63	3·07
Total	1,202	56·59	1,270	61·44

	1909	1908
Number of employees—		
White	4,972	4,745
Coloured	5,806	5,971
Total	10,778	10,716
Capital expenditure	£14,058,091	£13,933,498
Rolling-stock—		
Locomotives	325	336
Carriages	449	460
Trucks	3,946	3,556

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[1] Short tons of 2,000 lbs.

In the Engineer's report the following comparative Table is inserted:—

Railways.	Train-Miles per Open Mile.	Maintenance Cost per Train-Mile, including Renewals.	Percentage of Maintenance to Revenue.	Percentage of Maintenance to Total Expenditure.
		d.		
Natal	5,095	7·80	7·9	13·30
New South Wales . . .	4,232	10·00	12·49	21·00
Ceylon	3,784	11·00	14·50	..
Victoria	3,323	13·30	14·97	26·00
New Zealand	2,918	21·00	22·45	31·00
Central South African . .	2,807	14·00	9·60	20·00
South Australian . . .	2,693	13·00	16·21	29·00
Cape Government . . .	2,195	13·00	14·00	17·00
Queensland	2,113	12·83	18·79	32·00
Western Australian . .	2,081	10·85	12·29	19·05

Among other features which intensify this comparison the Engineer states that 12 per cent. of the total length is on 1-in-30 gradients, and that all traffic between Durban and Charlestown rises 13,599 feet and falls 8,200 feet.

This is the last Report of the Natal Railways as a separate system. C. O. B.

Indian Railways.

(Administrative Report by the Railway Board, 1909.)

All the railways in India and Burma are included. The money figures, except where otherwise stated, are in lakhs of rupees. (1 lakh = Rs.1,00,000; Rs.15 = £1.)

	1909	1908
Mileage 31st December—		
5-foot 6-inch gauge	16,309	
3 „ 3⅜ „ „	13,323	
2 „ 6 „ „	1,443	
2 „ 0 „ „	415	
Total	31,490	30,581
Sanctioned during 1909—		
5-foot 6-inch gauge	204	
3 „ 3⅜ „ „	398	
2 „ 6 „ „	493	
2 „ 0 „ „	11	
Total	1,106	
Gross earnings	4228·29	4011·79
Working-expenses	2484·57	2536·35
Net earnings. . . .	1743·72	1475·44

	1909	1908
Percentage of working-expenses to gross receipts	56·06	60·24
Train-mileage (in thousands)	128,260	127,881
Passenger-journeys „	329,380	321,170
Goods-tonnage „	60,900	62,400
Unit mileage of passengers (in thousands)	12,364,579	12,102,929
Freight ton-mileage (in thousands)	9,340,441	9,925,830
Average passenger-mileage	37·54	37·68
Average ton-mileage	153·37	159·07
	Pies.	Pies.
Average rate charged per passenger-mile	2·43	2·44
„ „ „ ton-mile	5·73	5·09
	Rupees.	Rupees.
Earnings per mile open	14,948	14,663
„ per train-mile	3·67	3·51
Working-expenses per mile open	8,380	8,833
„ „ per train-mile	2·06	2·11
Percentage of net earnings to capital	4·81	4·33
Number of employees—		
European	7,283	7,344
Eurasian	9,698	9,951
Native	498,722	508,188
Capital	42,983·20	41,191·71

	Gauge.			
	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.
Rolling-stock	5 6	3 3⅜	2 6	2 0
Locomotives	4,517	2,209	220	78
Carriages and vans	10,896	7,642	693	266
Trucks, etc.	95,247	46,429	2,881	998
Motor-cars	26	14	3	1

The Report is accompanied by maps which show a large mileage of projected lines referred to in one of the Appendixes.

C. O. B.

Railways of Japan.

(Revue Générale des Chemins de fer, Paris, 1910, vol. xxxiii, pp. 424-9.)

The first programme for the construction of railways in Japan was drawn up, with the help of the United States, in 1867. Negotiations were interrupted, however, by the Japanese Civil War of the following year. Soon after the war a loan of £1,000,000 was borrowed from an English financier for constructional purposes, and work under English engineers began on the Tokio–Yokohama line, the first train running in May, 1872. This period of construction continued up to 1885–86, during part of which the State and a private company were busy building independently. About 1884 the latter had completed 62 miles of railway. The second period extending to 1899 was one of private companies. In 1892 a law was passed which had the effect of separating the railways into two groups, viz., those urgently needed and required to be finished in 12 years at most, consisting of 1,900 miles, and those of secondary importance, of 2,525 miles, of which about

670 miles were owned by private companies. In 1894 a great impetus was given to railway building, so that in 1897 alone 466 miles were opened to traffic, and in the following year there were in the country 3,415 miles of railway, 760 miles of which were worked by the State and the remainder by private companies. The third period, dating from 1899, was remarkable for the number of amalgamations of companies, and for the fact that the Government, after much opposition, decided to purchase the private lines. At this period the State lines had a length of 1,490 miles and the private lines 3,266 miles. Recently the line connecting the extreme north with the south, a distance of 1,750 miles, has been completed. The lines generally are laid out with special regard to the defence of the country as well as to its development. Between 1900 and 1908 the railway-mileage has been increased by 1,200 miles, and the number of passengers from 114 to 141 millions, while the receipts have nearly doubled.

G. H. S.

Remedies for Landslides and Slips on the Kanawha and Michigan Railway. R. P. Black.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 877–83.)

The southern part of the Kanawha and Michigan Railway, for a distance of 93 miles, is on the east side of the Great Kanawha River, and for about one-third of this distance the line is close to the river-bank, on a hillside where there is practically no valley, the mountains rising directly from the stream. Owing to the character of the soil there is considerable trouble due to landslides and slips, the latter term being used to describe the movement of the embankment under the lines towards the river. Landslides occur in almost every case where the hillside has been cleared of vegetation. The Author describes the means taken to prevent a further occurrence of these slides and slips. In one of the worst places the method used for preventing landslides was to drive two parallel rows of piling 5 feet apart and 3 feet from centre to centre. The upper rows against the hill were backed with 3-inch plank, the front rows being driven against this brace to aid in supporting the upper row. A 10-by-10-inch stringer was placed against the upper row, and from this 8-by-8-inch braces were carried diagonally, at an angle of 45°, to the lower row of piles, and these were sawn off at the ground-level. Steel bands, with 1-inch rods to hold the two sets of piling together, were put on about 8 inches below the top of the brace-pile. The earth from the top of the piling was given a slope of 1½ to 1, the upper limit of the slide being about 135 feet above the lines. In the spring of 1908 the length of line affected by slips was 7,600 feet. A stratum of rock below the earth slopes towards the river, and immediately above this rock is quicksand mixed with fine clay. The slips were effectively dealt with in places by driving with a track-driver, two rows of piling, one on each side of the line, the piles being shod with steel. Maps show the extent of the slides.

R. S. B.

Electric-Railway Catenary-Trolley Construction. W. N. SMITH.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 849-84.)

This is a review of current practice in catenary-trolley construction, with a description of many details, such as steel poles, brackets, and hanger-rods; several designs of the latter are given, and it is stated that it is desirable to have at least one pivot-joint in the rod, either at the messenger or at the trolley-wire. The question of the material of the messenger-wire and of the trolley-wire is discussed, especially in connection with the expansion due to temperature. The temperature-range to be dealt with in the United States is 130° F., and it is stated that the minimum tension should not be less than 2,000 lbs. at the highest temperature, which entails, with copper-wire, a tension of about 5,000 lbs. at the lowest temperature. Photo-reproductions of several catenary suspensions are given, such as those on the Erie Railway, the Rochester-Syracuse Eastern Railway, and a new construction on the New York and Newhaven Railway, in which lattice steel posts bent inwards and forming an arch across four lines of track are used. It has been found that the increased tension required in the trolley-wire in catenary construction makes it unsafe to depend on a mechanical splice, and therefore bronze splicing-sleeves are used, into which the ends of the trolley-wire can be inserted and soldered. The old-fashioned trolley-wheel is unsuited for high speed, and has to be replaced by the sliding-bow. A rolling-contact is, however, desirable, and a design is given of the pantograph roller-trolley used on the San Francisco, Oakland and San José Railways. It is stated by the users that the life of these rollers is 67,500 miles.

H. R. S.

Superheated-Steam Locomotives. JULES DERY.

(Revue Universelle des Mines, Liége, 1910, pp. 213-51.)

From a practical point of view superheated-steam locomotives may be said to have come into use within the last 4 or 5 years. Previous to 1900 the difficulty of constructing heaters capable of reaching a sufficiently high temperature and the want of a suitable lubricant had proved insurmountable obstacles, and the attention of engineers, especially in England, was directed to higher pressures and compounding. In other countries, however, efforts to deal with the problem were not slackened. Wilhelm Schmidt, an engineer of Cassel, who was the first to introduce high superheating into practice, was engaged at the beginning of this century by the State of Prussia to apply his system to locomotives, and in a short time it was generally adopted on the Prussian railways. This example was speedily followed by the Moscow Company at Kazan and by the Belgian State railways. There are now about 4,000 locomotives in the world fitted with the Schmidt system.

Superheaters for locomotives may be classed in two categories:—those which utilize a portion only of the live gases of combustion, and those in which all the hot gases pass through the heater. To the former belong the Schmidt, Cole, Vaughan and Notken, and to the latter the Pielock, Slucki and Clench-Goldsdorf types of heaters. There are two different forms of the Schmidt type, that which is placed in the smoke-box and that which is fitted to the smoke-tubes. Experience has shown that the latter offers the following advantages over the former:—it is more simple in construction, has less weight (better distributed over the length of the boiler), is more easily accessible, can be more easily put together or removed, and its first cost is less. For these reasons this form has been adopted on the Belgian State railways. In the United States there have been produced various designs of heaters all derived from that of Schmidt. The first, the Schenectady superheater, made by the American Locomotive Company of Schenectady, New York, has been applied with satisfactory results to 55 locomotives of the Canadian Pacific Railway. A second, the Canadian Pacific Railway superheater, is a modification of the former, designed to remove some of the difficulties of upkeep and repairs which hindered the general adoption of that type. In 1906 this modified form had been applied to 77 of that Company's locomotives. It may be said that for engines of moderate power the superheating system has ousted the compounding, while it allies itself advantageously with the compounding system for engines of great power. The superiority of the superheated-steam engine is evident from the fact that, by compounding, the loss through condensation is reduced from about 35 to 20 per cent., while by superheating to 300° C. and above condensation is eliminated altogether. Tables of results of comparative trials made with Belgian locomotives show that superheating is very favourable to economy of steam and fuel. The Author describes the several types of superheaters and the modification of parts rendered necessary by the adoption of the system, and points out that the only suitable lubricant is a mineral oil capable of standing a temperature of 350° to 400° C. The article is illustrated by three Plates.

G. G. A.

Trials with a 2–8–0 Goods-Locomotive on the Italian State Railways. G. HEISE.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 128–33.)

The article details the results obtained on the trial runs made on the Pistoia-Pracchia section of the Italian State Railways with a 2–8–0 compound locomotive,[1] having a heating-surface of 2,200 square feet and a grate-area of 30·5 square feet. A boiler-pressure of 235 lbs. per square inch was maintained during each test. The high-pressure cylinder was 19¼ inches, the low-pressure 29½ inches in diameter; diameter of driving-wheels 4 feet 6 inches. Full dimensions and a drawing of the locomotive are given, and the results of 175 trials of locomotives of the above type are recorded in tabular form. The steepest parts of the railway have inclines of 1 in 40, and the sharpest curves to be negotiated are 985 feet radius. The necessary instruments for carrying out the tests were attached to the locomotive, the regulator being provided with a device enabling the observer to note the extent to which it was open. An adjustable exhaust-ring was provided for varying the vacuum in the smoke-box. The fuel consisted partly of Newport coal, having a calorific value of 14,000 B.Th.U., and partly of Italian compressed-coal of 13,400 B.Th.U. Full details of the steam- and coal-consumptions are recorded, both being considered low. The cylinder-diagrams, which are reproduced, show that the work in the cylinders was well and evenly distributed. With a load of 528 tons a velocity of 28½ miles per hour was attained up an incline of 1 in 100.

C. J. G.

Freight-Train Resistance. E. C. SCHMIDT.

(Journal of American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 679–721.)

The object of the tests described in this Paper was to determine the resistance of freight-trains in ordinary work, and to find if any relation existed between the train-resistance and the average car-weight. The speeds were 5 to 35 miles per hour; the minimum weight of a train was 740 (short) tons, with an average weight per car of 16·12 tons; the maximum was 2,908 tons, with an average weight per car of 69·91 tons. Some thirty-two trains were tested, the road on which trials took place being representative of good main-line construction. The observations were taken by a dynamometer-car, of which full particulars are given in an Appendix. The results are plotted on curves giving the train-resistance in lbs. per ton compared with the speed, and also with the average weight per car. As regards the latter, there is a distinct diminution in train-resistance as the weight of the car increases; thus, approximately, at 16 miles per hour it was 8 lbs. per ton for 20-ton cars, 5 lbs. per ton for 40-ton cars, and 3½ lbs. per ton for 70-ton cars. The results of all the trials have been averaged, and a Table has been prepared therefrom giving the train-resistance for cars weighing 15 to 75 tons and for speeds of 5 to 40 miles per hour.

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[1] Described in the *Zeitschrift des Vereines deutscher Ingenieure*, 1907, vol. li p. 1885.

H. R. S.

Mechanical Principles of the Brennan Mono-Rail Car.

H. T. Eddy.

(Journal of the Franklin Institute, Philadelphia, 1910, vol. clxix, pp. 467-85.)

The action of the gyroscope as applied to the Brennan mono-rail car is explained in a comparatively simple manner. As a preliminary an analogy is established between a mass rotating horizontally in a circle attached to a central point by a string and a gyroscope rotating on a horizontal axis supported at the central point, the gyroscope replacing the mass. It is shown that the momentum of the rotating mass corresponds with the angular momentum of the gyroscope, the tension of the string to the torque of the weight of the gyroscope multiplied into the distance from the point of support, and the angular velocity of the rotating mass to the precession of the gyroscope. These three quantities are directed at right angles to each other, and the Author suggests that it greatly assists the student of the mono-rail car to use a model made of cork, into which three pins of different kinds are stuck, one along the axis to represent the angular momentum, and the other two at right angles to represent respectively the torque and the precession. The arrangement of the gyroscopes on the Brennan mono-rail car is shown diagrammatically, and by the help of the preceding considerations and the model cork it is explained how a car rights itself if the weights are displaced and how it behaves in going round curves. It is also shown that one gyroscope would be sufficient for all purposes except going round curves, for which two are needed, revolving at the same speed but in opposite directions. It is shown that there is no practical probability of circumstances arising which might cause danger and upset the car.

H. R. S.

Motor-Vehicles (Great Britain and Colonies). T. H. RICHES.

(Bulletin of the International Railway Congress, London and Brussels, 1910, vol. xxiv, pp. 1495–1574.)

Replies received from twenty-one railways in Great Britain, Africa, Australia and India show that motor-cars are used in ordinary traffic. Small boilers of the locomotive type and straight-link motion are usual for steam-cars. The speeds are 25 to 45 miles per hour, and gradients as steep as 1 in 38 can be worked on. The second stage of development is when one small car takes another with it when necessary, the third stage when the self-contained motor-car has a carriage attached, and the fourth when a small locomotive is placed between the carriages. Appendixes give the replies of the North Eastern Railway as to petrol- and steam-cars, and—for all the railways which have replied—the points between which cars are run, distances, intermediate stops, and number of cars in daily use.

W. H. B.

International Sleeping-Car Company, 1872 *to* 1909.

(Revue Générale des Chemins de fer, Paris, 1910, vol. xxxiii, pp. 3–44.)

The first train de luxe in Europe was the Orient Express from Paris to Vienna, which began running in June, 1883. Similar trains now run on all the great trunk lines and the operations of the Company extend over parts of three Continents.

The early carriages, about 26 feet long, and divided into three compartments, one of which was reserved for ladies, had a central corridor, and were adapted for day and night use, were plainly constructed, lighted by colza-oil lamps—later by the "Silber" lamp—and heated by hot air or steam and even by briquettes. Bogie-carriages did not come into general use until about 1883. The carriages were of three types, viz., 1st- and 2nd-class combined day- and sleeping-cars, 1st-class sleeping-cars, and restaurant-cars. Continuous improvement took place in size and equipment until the present four- or six-wheel bogie-car was reached, with its excellent lighting, heating, sanitary and restaurant arrangements, and its great length of 58 to 78 feet.

The Company has thirteen large repairing- and construction-shops scattered over Europe, employing 2,000 men, through which some 1,500 cars of various kinds pass in the year. The rolling-stock is composed of 750 sleeping-cars, 520 restaurant-cars, forty saloons, ten refreshment-buffets and 180 wagons. The more important workshops are at Paris, Zossen (near Berlin), Vienna, Slykens (near Ostende), Moscow and St. Petersburg.

G. H. S.

Rail-Joints (France, Belgium, Italy, Spain and Portugal).

CHATEAU.

(Bulletin of the International Railway Congress, London and Brussels, 1910, vol. xxiv, pp. 1427–64.)

The Paper is divided into (1) reduction in the numbers of joints, and (2) strengthening of joints. Typical lengths of rails vary between 54 feet 2 inches and 78 feet 9 inches, the most general length being 59 feet, and rails are usually shorter the hotter the climate. No increase in length is proposed generally. Disadvantages of long rails are difficulties in carriage, handling, straightening, arrangement at curves, and obtaining sufficiently strong fish-plates. Limiting lengths suggested vary between 32 feet 10 inches and 72 feet 2 inches. It is probable that for several years 65 feet 7 inches will continue to be used. It is feared that welding will cause the track to warp. The general tendency is to stiffen the joint by bringing the sleepers closer together. Several railways, especially in Spain, have abandoned support for suspended joints. Descriptions of various joints are given. The rails taken yearly out of main lines have still a long life, and most railways have applied to old rails solutions similar to those tried with new rails. The French Northern and Western railways have tried to remedy deformations by cutting the ends, and the French Western railway has straightened about 10,000 rails, which have been in use 25 to 30 years. The lengths of these were 19 feet 8 inches and 26 feet 3 inches, and the maximum permanent vertical deformation was as high as $1\frac{9}{16}$ inch in the case of the longer rails. The Author investigates the phenomena of curvature of rails. The cost of strengthening each rail separately is about 8·77 pence per yard. The use of thin wedges causes broken fish-plates, and some railways replace worn fish-plates by new ones. The French Midi Railway practise "retaliation" of the joints, placing the joints a little higher than the middle of the rails, and increasing the packing of the joint-sleepers.

W. H. B.

Rail-Joints (All Countries except France, Belgium, Italy, Spain, Portugal, Austria-Hungary, Rumania, Bulgaria, Servia, Turkey, Egypt, and English-speaking lands). **BLUM.**

(Bulletin of the International Railway Congress, London and Brussels, 1910, vol. xxiv, pp. 1701-48.)

Rails have been lengthened generally, lengths of 39 feet 4 inches, and even 49 feet 3 inches having been introduced. Normal lengths vary between 29 feet 6 inches and 49 feet 3 inches. Only the Württemberg State and Java railways propose further increase. As the maximum admissible length, 49 feet 3 inches or 59 feet is quoted. The maximum length is limited by the gap between rail-ends, which should not exceed $\frac{25}{32}$ inch, also by cost and difficulties in carriage and laying. Special arrangements for slacking bolts and for expansion are only used on the German and Norwegian railways. Welding is not customary. There has been increased use of better ballast and of bottom ballast. No special ballast is used at joints unless the track is badly ballasted. The depth of ballast has also been increased. Details of rail-lengths, climates, ballast, weights of locomotives and trains, and speed of trains, are given in Table I, axle-loads and wheel-bases in Table II, and a summary of the conclusions as to rail-lengths at previous Sessions of the Congress is also given. All railways are trying to improve joints by strengthening the fish-plates, which are made larger also, and, where suspended joints are retained, by decreasing the distance between the joint-sleepers; some railways also diminish the distances between adjoining sleepers also, more particularly on the facing side. Joint-sleepers are sometimes laid in contact, and at times a sleeper is laid directly under the joint. Table III gives particulars of the most recent forms of joints. The three-sleeper joint of the American type has not proved satisfactory. Various improvements for joints are mentioned, amongst others reduction in sleeper-spaces, planing to remove the difference in height of the rail-heads, shortening rails by half a fish-plate length, and measures to prevent the wear of fish-plates and seats and the movement of fish-plates.

W. H. B.

Cause of Rail-creeping on the South-Holland Electric Railway.[1]

T. J. W. van Rossem.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 362–6.)

A close study of the cause of rail-creeping on this line disclosed the fact that it must be largely ascribed to the spring-suspension of the electric motors. A mathematical analysis shows that a small vibration of the motors results in a traction-effort of 2,240 lbs. on the left wheel and 1,375 lbs. on the right wheel at one moment, and 1,105 lbs. on the left with 2,500 lbs. on the right wheel at the next moment. This variation in traction-effort causes one of the two wheels to slip, giving rise to rail-creeping. The excess in effort of the right over the left wheel is 530 lbs. greater than the excess of the left over the right wheel, and this causes the right-hand rail to creep in advance of the left-hand rail. These alternations in traction-effort set up a twisting-moment, causing the coach to oscillate between the rails, and thus intensifying the creeping action. This oscillation and the alternate slipping of the right and left wheel on the respective rails are the causes of rail-corrugation[1] on this line. The article is illustrated.

H. C. A. T.

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[1] *See also* Minutes of Proceedings Inst. C.E., vol. clxxiv, p. 407.

Autogenous Welding in a Railway-Shop. H. W. Jacobs.

(Railway Age Gazette, New York and Chicago, 1910, vol. xlviii, pp. 1535-9.)

The application of this process to railway-shop practice is new, the apparatus now on the market being expensive and too small for railway work. An American railway having itself installed the process on a suitable scale, the Author is enabled to say that great saving may be accomplished by its adoption, and that development should be along the lines of reducing the cost of the manufacture of the gases for large central plants. A comparative Table of costs is given of removing and replacing 2-inch boiler-flues by the usual method of rolling and beading, and of welding flues by the oxy-acetylene process. This shows the latter to be about 5 per cent. higher than the former, but the life of the flues is extended almost indefinitely, or at least until removed for some other cause than leaking at the flue-sheet joint. An incidental advantage is the possibility afforded of giving value to otherwise practically worthless scrap fire-boxes by cutting them up into sheets, saving about 19*s*. 7*d*. per ton.

C. O. B.

Pressure, Resistance and Stability of Earth.

J. C. Meem.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 653-89.)

The Author describes the results of practical experiments made to ascertain earth- and sand-pressures, and the application of the deductions to works carried out. After describing the apparatus employed, which consisted of receptacles of large size containing orifices and means for measuring the effects of the pressure of the contents, he compares the conclusions with observed phenomena on works such as the Bay Ridge tunnel-sewer and the Pennsylvania tunnels. Experiment and observation point conclusively to the fact that pressure is transmitted laterally through ground along or nearly parallel to the angles of repose, or, in the case of rock or stiff material, along a line which may be taken as approximately 45°. The Author believes also that the question of confined foundations, in contradistinction to spreading foundations, is important. He has found that, in soft ground, results are often obtained with small open caissons sunk to a depth of a few feet and chained out and filled with concrete, which offer much better resistance than spreading the foundation over four or five times the equivalent area. The bearing-value and friction on piling also form the subject of comment in the light of the experiments.

R. S. B.

Silo for Grain-Storage at Bremen. E. Lufft.

(Beton und Eisen, Berlin, 1910, vol. ix, pp. 227-9.)

This building is said to be noteworthy both on account of the material used for the cells and for the foundations in reinforced concrete. The designs were prepared by Messrs. Amme, Giesecke and Konegen, of Brunswick, who have employed this method of silo-construction in America. The cells, twenty-four in number and constructed of iron cylinders, are 90·21 feet in height by 16·4 feet in diameter, and when filled with heavy grain were calculated to weigh 12,000 metric tons. The structure rests on made ground, tipped about 8 years ago, and consisting of fairly uniform layers of sand. It had been ascertained from other buildings in the vicinity that it was not safe to load the subsoil to the extent of more than 35·5 lbs. per square inch, and for this reason the concrete foundation was spread out for a width of 6·56 feet all round the outside of the silo with external buttresses so as to constitute a solid slab, with an ultimate bearing of 30·8 lbs. per square inch over the entire area. The construction of the reinforced-concrete foundation is explained by reference to diagrams. It was designed both in the longitudinal and transverse directions as a slab in which the concrete was not stressed beyond 497 lbs. per square inch, or the steel reinforcement to more than 14,223 lbs. per square inch in section, and no shearing-stress exceeded 56 to 71 lbs. per square inch. The silo-cylinders as well as the inter-spaces were used for grain-storage, and the whole were covered with a roof of reinforced concrete, of which full details are given, with photographs showing the method of inserting the steel reinforcement.

G. R. R.

Paint for Iron and Steel. F. Neal.

(Journal of the Association of Engineering Societies, Boston, Mass., 1910, vol. xliv, pp. 267-71.)

The claim that linseed oil is the life of paint does not necessarily apply when iron and steel are the materials to be coated. Not only is the pigment to be mixed with the oil necessary, but also some other vehicle to offset certain characteristics of the oil. In painting wood there is a more or less absorbent surface, which is not the case with iron. The oil must make its own bond in such a way as to form a thin even coating not easily scraped off, and this cannot be done by the use of linseed oil alone. The Author describes experiments made to ascertain the best composition for iron and steel paints, and comes to the conclusion that for such purposes the paint should consist of linseed oil in combination with the pigment, diluted with a volatile liquid such as naphtha. The least possible amount of oil should be used in the priming coat, gradually increasing the percentage of oil and decreasing that of the pigments in succeeding coats, directly the opposite of that which would be best for wood such as white pine.

R. S. B.

Diagrams for Designing Reinforced-Concrete Beams.

F. H. Carter.

(Engineering Record, New York, 1910, vol. 61, pp. 786-8.)

These diagrams enable the various dimensions, including the spacing of the rods, certain data being supplied, to be read off, a special example being given in the text explaining their use. A thick hard line is carried on successively through the seven diagrams following the computation of the particular case illustrated, thus indicating the procedure in others. Additional diagrams give the spacing in inches for square bars and their proportional area to that of the concrete. The diagrams are based on the straight-line formula, making no allowance for tension in the concrete, the ratio of the moduli being taken as 15.

C. O. B.

Reinforced-Concrete Wharf on the Guadalquivir at Seville.

J. M. Zafra.

(Revista de Obras Publicas, Madrid, 1910, pp. 209-16.)

To serve the mineral traffic of the Port of Seville, which is rapidly increasing, there are (1) three timber piers built by the river Board of Works where 3,000 tons can be loaded daily by iron barrows of large capacity; (2) two reinforced-concrete loading-stages of the Cala Mines[1] of 6,000 tons capacity, and (3) the wharf, of 2,000 tons daily, the subject of this extract, belonging to the Compañia Gaditana de Minas. This company's railway from the mines at Aznalcóllar to the Guadalquivir, 22 miles long, serves an important district where copper pyrites is worked, and, from considerations of cost, was constructed across the Guadalquivir valley on the level. The wharf therefore is at the general level of the valley, 17·5 feet above low water, and parallel to the river, into which it projects so as to obtain sufficient depth of water for vessels of 4,000 to 5,000 tons. Down the centre runs the 1-metre-gauge railway, and, on the same axis, another line 16 feet 4·8 inches from centre to centre of rail to carry the cranes, one of 10 tons and others of 5 tons ordinary lifting power, and 39 feet 4·4 inches radius. The wharf is reached by a curve and countercurve of 328 feet radius with a depôt-siding on shore for sixty wagons. It is 17 feet 4·6 inches wide and 533 feet long, of which 167 feet are in the entrance curve, and provides standing room for twenty wagons; the usual accessories are also of reinforced concrete; the fenders, 36 feet apart, are of iron, independent of the wharf from which they stand out 27·5 inches. The structure is carried on two rows of reinforced-concrete piles 16 inches square, 10 feet 10 inches centres, spaced 18 feet longitudinally, and driven 26 to 31 feet below the level of low water. Above this level the section of the piles is reduced to 14 inches square, and they are braced in both directions at a height of 4 feet; there is also secondary transverse bracing between the longitudinal beams of the floor to meet possible torsional stresses from the load. The piles are reinforced by four 4-inch deep rolled joists at the angles, the flange forming a bevel at the corners; these are bound with No. 6 standard gauge wire 10 inches apart except for 6 feet at the top, where the spacing is reduced to 4 inches. Where the upper part of the pile decreases to 14 inches the rolled joists are replaced by four 1-inch diameter bars. Along the top of each row of piles for the whole length of the wharf runs a beam 31·5 inches deep by 14 inches wide connected to the other by the floor, which is 8 inches thick; this carries the metre-gauge railway, the 56-lb. rails of which are simply buried in longitudinal beams, except on the curve where, besides tie-rods, the rails are bound to the floor by the reinforcement. The junction between the beam and the floor is filled in to form a gusset at an angle of about 60° with the vertical. The piles being 10 feet 10 inches apart, the remaining 6 feet 6·6 inches of the total width of the floor overhangs the supports on either side to the extent of 3 feet 3·3 inches, near the extreme end of which, and 2 feet 9 inches from the centre of the pile, is carried the 84-lb. rail of the crane-railway. The depth of these cantilevers is 16 inches at the root, the upper face being inclined slightly from the outer edge of the pile to the rail, which is 8 inches higher than that of the metre-gauge railway in the centre. The minimum thickness is 11 inches, the outer end is 16 inches thick and octagonal in form, containing, buried within it, besides the 84-lb. rail, an inverted 28-lb. rail also immediately below it bound to the former by ½-inch reinforcement, 16 inches apart, and bars of the same diameter support the 84-lb. rail laterally counteracting the tendency to turn over.

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[1] See *Engineering*, 1907, vol. 84, p. 572.

The Author, anticipating criticism, justifies the position of the supporting piles, arguing that his plan of carrying the outer railway on cantilevers and the metre-gauge line between the supports placed at 10 feet 10 inches centres is simple and economical, starting from the hypothesis that in a wharf of the width here dealt with more than two lines of piles would be inadmissible. The electric cranes are erected on a lower body with four legs, each carried by two wheels, which span the railway and wagons. An unloaded crane weighs 74·5 tons and exerts on each foot of the four legs pressures of 13·27 to 25·79 tons; with a load of 5 tons these pressures are 18·50 to 21·33 tons; and with a 10-ton load 13·57 to 31·78 tons.

The locomotives are eight-wheeled and weigh 36 tons on a wheel-base of 13 feet: the mineral wagons carry two tipping-boxes each with 5 tons of mineral. Under the most unfavourable circumstances the load on a pile at the level of low water may reach 58 tons, corresponding to 600 lbs. per square inch at the point where the section is reduced from 16 to 14 inches. The piles, 43 feet long, were built on the river-bank and launched, when a month old, into the water, where they were picked up by the gear on the piling-stage. Each pile was driven until it resisted ten blows from a height of 24 inches with a maximum penetration of 0·2 inch, equivalent to a load of 62·5 tons.

In practice the actual work done averages 100 tons of mineral per hour. The cost of the works was £8,072. The cement employed was Spanish.

C. A. F.

Action of Sea-Water on Portland-Cement. J. M. O'Hara.

(Engineering Record, New York, 1910, vol. 61, pp. 677–8.)

Premising that concrete has remained intact in sea-water for lengthened periods, while in other cases rapid destruction under similar conditions has taken place, the Author presents a series of tests in order to select a cement for efficient use. He gives the chemical analysis of the salts of sea-water as $MgSO_4$, $MgCL_2$, $NaCl$, and $CaSO_4$, and the essential components of Portland cement as $3CaO, SiO_2$ and $2CaO, Al_2O_3$; while Fe_2O_3, MgO, SO_3, K_2O, Na_2O, CO_2 and water always occur in appreciable quantities. The specimen pats of various brands were allowed to set in moist air for 24 hours, and then immersed in a solution containing the following percentages of the different salts in the order given above, viz. 0·9, 1·4, 11·5, and 0·6 with 85·6 of water, when, after 90 days, many pats had disintegrated. Strong solutions were then made and six brands of cement are represented in each of two Tables, one referring to neat and the other to 1·3 briquettes, the tensile strength after sea- and fresh-water immersion being given for 7 and 28 days, 2, 3, 6, 9 and 12 months.

The subject is discussed in the light of these results, and under the separate headings of three classes of failure, viz., swelling and bursting, loss of strength, and softening, and the Author explains the reasons for the difference of some of his results from those corresponding to the generally accepted theory in these particular cases. A diagram gives results in the case of one brand, showing a distinct decline in strength after 2½ months' immersion in sea-water.

C. O. B.

Action of Sea-Water on Concrete. J. A. van der Kloes.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 271-3.)

The experiments of A. Poulsen and W. Czarnowski appear to show that the deterioration of concrete is mainly caused by the mechanical action of frost and not by chemical reaction of sea-water on cement. The Author gives photographs showing the growths of magnesia salts on the concrete, from which he draws the conclusion that these statements are not entirely correct. These growths are commonly called concrete tulips, owing to their remarkable resemblance to that flower. The Author's investigations show that, where deterioration of concrete has taken place, it is wholly attributable to the bad mixing and proportioning of the various aggregates used. The filling capacity of the aggregate should be determined by placing a measured volume of the aggregate in a tank and measuring the volume of the interspaces by means of water. The best material is that which requires least water. Any concrete structures showing the formation of chalk on the surface contain too high a proportion of chalk and a poorer cement should be used. The article concludes with a series of proportions to be used for various concrete marine structures.

H. C. A. T.

Electric Irrigation-Pumping in Kansas. E. C. Reybold, Jun.

(Electrical World, New York, 1910, vol. lv, pp. 1532-3.)

The article describes a high-tension electric-transmission scheme and series of pumping-plants erected for irrigation purposes on about 5,000 acres of land in the Arkansas Valley. An underflow at 7 feet to 18 feet below ground-level is available. Generating-plant of 375-kilowatt capacity, driven partly by suction-gas and partly by oil, feeds a transmission-line 15 miles long, operating fourteen pumping-plants each irrigating a 320-acre tract; the number of pumping-stations is to be shortly increased. Cipoletti-weirs measure the flow of water and retain water for priming. The total cost of installation is about £5 5*s*. per acre of land irrigated, and the cost of energy per acre-foot of water pumped is estimated as less than 4*s*. 2*d*.

J. A. S.

Hydraulics of the Chagres River. General H. L. Abbot.

(Engineering Magazine, London, 1910, vol. xxxix, pp. 377-84.)

The sufficiency of the Chagres River as the principal source of supply to the Panama Canal, with its extensive lockage, is the subject of this article. The characteristics of the river and its basin, the rainfall and the floods, from several years' observations, are discussed. The division of the supply into percolation, including subterranean chemical changes, evaporation and outfall, is referred to, and a Table records observations in each of the years 1898 to 1909 inclusive, of the rainfall and the outfall above Bohio, demonstrating the adequacy of the supply. The conclusion is arrived at that, of the total volume, about one-third escapes directly through the beds of the streams, another third finally reaches the sea by the same channels as ground-water, and the remainder returns to the atmosphere by evaporation.

C. O. B.

Pontoon for Subaqueous Rock-Boring.

(Le Génie Civil, Paris, 1910, vol. lvii, pp. 87–8.)

A rock-drilling pontoon of novel design has been constructed for the Port Authority of Triest; it consists of a vessel 59 feet long by 20 feet beam practically rectangular in form and carrying on one side an overhanging gallery, which serves as the working-platform for two electrically-driven percussion-drills. The notable feature of the vessel is that it is furnished with sliding metal legs on which it can be made to rest while drilling is being done. The four legs, one near each corner of the pontoon, are each pierced with a series of rectangular holes so arranged that any leg, on being lowered to the sea-bed, can be fixed to its respective crosshead by the simple insertion of a cotter.

Coupled to each crosshead are two screw pulling-jacks with their nuts firmly bearing at about deck-level against the guiding bracket, within which slides the leg on which they operate. The nuts are capable of being rotated by power, supplied through gearing, from the main engine in the hold; thus the pontoon can be raised clear of the water, and again lowered when required. The legs are each made up in sections so as to be capable of extension when work is to be carried out in deep water, the maximum length being 42·6 feet. During the tests the pontoon was "freed" to the extent of 2·3 feet and remained in this condition without incident for 3 days. Power is supplied to each drill by an electric motor which, through the intermediary of gearing, actuates a crank-shaft, and this by means of a connecting-rod effects, once in every revolution, the compression of a helical spring. The energy thus stored when suddenly released causes the downward blow. The number of blows per minute is normally 380 and the stroke 2·35 inches. The drills are guided near the surface of the ground and can be set at any inclination within a range of 15° from the vertical. The spindles can be fed forward to the extent of 8·2 feet, and holes of 6·56 feet in depth by 3·15 inches in diameter can be drilled in moderately hard limestone at the rate of 3·28 feet per hour. Each drill absorbs 2·5 kilowatts and a single-cylinder engine, developing, at 320 revolutions, 28 HP., supplies the whole of the power needed, except such as is required for manœuvring the vessel.

I. C. B.

Lengthening of the Castigneau Graving-Dock at Toulon.

MAROGER.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xliii, pp. 93–113.)

An extension at the upper end of the Castigneau graving-dock, furnishing an addition of 82·33 feet to the length, has recently been completed. The "bottom" at Toulon has always been regarded as of so porous a character as to preclude the carrying out of dock-works in the dry, and the method employed for effecting the lengthening of the dock was chosen partly to obtain more accurate particulars of the foundation material, information which it was desirable to procure in view of larger works contemplated. For the foundations, three masonry caissons were sunk in the open from quay-level to a depth where the infiltration of water became so excessive as to necessitate the use of compressed-air. The caisson supporting the head of the dock is pentagonal in plan, and has an area of 720 square feet, while the other two, on which the respective side walls rest, are quadrilateral, each covering an area of 630 square feet. A wrought-iron cutting-edge, having a depth of 6·56 feet, was firmly attached to each of the caissons, and, at a height of 13·94 feet above the lower edge of this, a strong platform of iron-work was provided to form the top of the working-chamber when sinking was being carried on under air-pressure. The wing caissons were carried down to a depth of 40·28 feet, and that at the head to a depth of 36·90 feet below low-water level, and all were sunk simultaneously, the added rings of masonry being allowed to set for 15 days before being brought into contact with the ground. When sunk to their full depth, the working-chambers were filled with concrete, and the spaces between the head and wing caissons, and also between the latter and the old work, were closed with plugs of concrete. The spaces in question were arranged each of sufficient width to admit of a diving-bell being introduced, by the agency of which the excavation was conducted and the respective plugs deposited. When the joints were thus sealed, the area enclosed was excavated in the dry and the caissons were cut down where necessary to admit of the dock-floor being laid. The foundation material was found to be by no means so treacherous as had been anticipated, but the area, to which the observations were confined, was insufficient to enable general deductions to be drawn. A Table gives the cost of all the items in the work.

I. C. B.

Reinforced-Concrete Dock-Works at Ipswich.

(Concrete and Constructional Engineering, London, 1910, vol. v, pp. 419-27.)

A description is given of the reinforced-concrete extension works to the timber quay at Ipswich, erected in 1880. The new deep-water quay, 27 feet wide throughout, with accommodation for a railway-line, consists of thirty-two bays founded on reinforced-concrete piles spaced 10 feet apart, except at one end, where the distance is 10 feet 6 inches. The front row of piles are each 14 inches by 18 inches and those at the rear are 14 inches square. The piles are driven down into the gravel and are calculated to sustain a load of 30 tons due to the rolling-load of a 27-ton shunting-engine and 2½ cwt. per square foot as a distributed load on the decking. Sheet-piling is employed keyed up with cement-grout to form a continuous diaphragm from end to end of the wharf, serving the purpose of a retaining-wall for the filling deposited on the landward side. Considerable underpinning had to be carried out beneath the front walls of the public warehouse with a frontage of 121 feet to the dock. The method of executing this work is explained by a section. The average length of the piles was 30 feet, and these were reinforced by four steel bars braced by transverse links at short distances apart. Considerable difficulties were experienced in driving the piles to the required depth through the gravel slope, which overlies a compact bed of chalk and clay, forming, with variations of level and solidity, the bed of the dock, but by continued driving the piles were finally sunk, some of them having received as many as 2,000 blows from a 2-ton monkey. The details of the cross ties and the construction of the warehouse-floor and quay are explained by drawings.

G. R. R.

Christiania Harbour-Extensions. O. STEENSGAARD.

(Teknisk Ugeblad, Christiania, 1910, vol. 57, pp. 308-13.)

After an account of the extensions and the general features of the harbour, the Author describes the various works in detail. There is generally only a slight rise and fall of tide and the harbour is well protected against heavy sea. The bottom consists of rock generally covered with soft material in varying thickness and incapable of carrying the weight of quay-walls. The line of quays has therefore been chosen as far as practicable to follow the contour of the rock to secure a sound foundation by dredging, blasting or piling at moderate depths. The practice adopted was to uncover the rock by dredging to a depth of about 56 feet below low water. With a height of quay-wall of 25 feet, 31 feet of stone filling was used. Where the depth to rock was greater, dredging was carried down to 33 feet and a layer of gravel 6 feet thick was deposited on the dredged surface; piles were then driven under the site of the wall and about 50 feet behind its inner surface, i.e., a total width of 70 to 75 feet. The length of piles is 30 to 50 feet and they are placed 30 inches apart under the wall and 40 inches apart behind it. The piles were cut at the proper level and a 24-inch layer of concrete was cast round their heads and levelled to form the seating of the quay-wall. Where stone-filling on the rock was used, levelling was effected by means of small broken stone or coarse ballast, proper allowance being made in height and time allowed for settling. The quay-wall was then built over low water and loaded in excess of subsequent working-load until no further settlement occurred, when the filling-in at the back was proceeded with. The quay-wall was at first built by means of timber caissons in which concrete was deposited, but this method was subsequently abandoned in favour of concrete blocks cast on land and deposited by a floating-crane, which was found more satisfactory in every way. The wall over low water about 7 feet high is faced with granite and finished with a 24-inch coping. Railway-lines and sheds are provided, as are also a few electric cranes, but as there is practically no tide, ships prefer generally to use their own winches.

L. S. Z.

Methods of Lighting Employed in Lighthouses. J. BÉNARD.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 123–65.)

After referring specially to the work of Fresnel, the Author describes the transmission of light through the generating profile, fixed and turning lights, different groupings of lenses and the arrangement of flashes. A list of the different orders of apparatus with the focal lengths, and an illustrated description of the apparatus in use at various lighthouses are given; also particulars of the use of vegetable oil, with a Table of consumption and intensity in the various orders of apparatus. Mineral oil was only used with a wick and lower reservoir between 1857 and 1870, after which burners with multiple wicks came into use, petroleum lamps with permanent light being introduced in 1892. Descriptions of Bourdelle's constant-level lamp, of the apparatus in use in England, and of incandescence by petroleum vapour and by injected oxygen are given. The methods of lighting described are ordinary gas, oil-gas, acetylene, acetone and electric arc. The progress in lighting by various oils and gases between 1790 and 1902, and by electricity between 1863 and 1901 is shown by diagrams. In conclusion, the Author discusses the use of Nernst lamps and the possibility of the adoption of Herzian waves.

W. H. B

Mechanical Stoking for Steam-Boilers. ÉMANAUD.

(Le Génie Civil, Paris, 1910, vol. lvii, pp. 69–84.)

Hand-firing competitions have shown that the personal factor enters largely into the question of stoking. One of the most important desiderata for economical working is the careful regulation of the air-supply. The quantity of air required to ensure perfect combustion can be calculated to a nicety, but in practice 40 to 70 per cent. over the theoretical quantity has to be introduced, owing principally to the imperfect manner in which it can be brought into contact with the fuel. While a paucity of air leads to waste by reason of imperfect combustion, an excess gives a like result owing to its abstracting from the fire the heat required to raise its temperature to that of the escaping gases. With gaseous, liquid, or, if in a state of extremely fine subdivision, even solid fuel, the regulation of the air-supply is comparatively simple as the fuel can be fed continuously; but when ordinary coal is used, at any rate if hand-firing be adopted, the simplicity at once vanishes, as the fuel-supply is then intermittent, while the inrush of cold air whenever the firing-door is opened is of itself sufficient to upset the working conditions. Uniformity in temperature is of the greatest importance, owing particularly to the fact that dissociation of the various gases produced by combustion takes place in some cases at by no means high temperatures. These considerations point to the probable gain which would follow the substitution of mechanical for hand-firing, and it has been generally admitted that economy is effected in large installations by this substitution. It has been claimed, however, that such economy has been chiefly, if not entirely, due to the saving in wages, a saving which would not apply in the case of single boilers. The Author shows the latter view to be erroneous, as the economy in fuel alone is sufficient in most cases to recoup the cost of a mechanical stoker in about 2 years. In an experiment conducted by the Association Lyonnaise des Propriétaires d'Appareils à Vapeur, on a boiler with a heating-surface of 1,937 square feet, the net saving over hand-firing was 11·25 per cent. The various apparatus for indicating perfectness of combustion are even more valuable with mechanical than with hand-firing, as the defects they show can be more readily remedied.

I. C. B.

Dutch Specifications for the Strength of Steam-Boilers.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 388–91.)

This article contains particulars of the stipulations set forth in connection with the testing of boiler-plates with reference to thickness, design and material, and also the design, arrangement and dimensions of safety-valves. The formulas to be used are those recommended by the Special Commission whose report was published in 1908.[1] The specifications are divided into six sections: (1) External pressures; (2) Internal pressures on cylindrical surfaces; (3) Formulas for flat surfaces, with special reference to stays and gusset-plates; (4) Formulas to be used for curved top, bottom and front plates with and without flanges; (5) Further details for calculating the size of anchor-bolts, tension-rods, tube- and bridge-stays: (6) Calculation and specification of safety-valves.

H. C. A. T.

Application of Rotary Blowers to High-Furnaces. A. Gouvy.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 342-50.)

The case considered is that of a rotary blower worked by an electro-motor, the electric motive power being produced by a central station with gas-motors. The advantages claimed are concentration of all the gas-motors in a single room, decreased bulk, simplicity of installation, a possibility of working each furnace with a special blower, reduction in maintenance and cost of lubrication, and regularity of pressure and of the air delivered to the furnace. The Author discusses the working of the furnace and blower, and to ensure proper regulation advises the use of (1) registering anemometers, (2) Pitot tubes, more especially the Schultze-Dosch apparatus, (3) a metal disk, whose displacement is noted on a dial, placed in the force-pipe, or (4) the Rateau multiplicator, the application of which to turbine-blowers is described in the Bulletin de la Société de l'Industrie minérale.[2] The Paper concludes with detailed statements of the economy of the installation and a comparative Table of the combinations of blowing-apparatus applicable to a high-furnace of 98½ tons per 24 hours.

W. H. B.

Regulation of Combustion in Furnaces. Eugène Grandmougin.

(Le Génie Civil, Paris, 1910, vol. lvii, pp. 4-7.)

For determining the perfectness of combustion in a furnace it is admitted that the best process is that of analyzing the gases escaping from the chimney, but unfortunately the apparatus required for this is costly, easily broken and difficult to use; some types do not give continuous records, while others, viz., those which depend on the determination of the specific gravity of the mixed gases, are liable to error, owing to the extremely minute difference which a small addition of carbon-dioxide causes. For general daily control it is desirable to equate the coal consumed to the water evaporated, account being taken of the pressure and consequent temperature of the steam. This method is simple, though not very accurate by reason of the varying quantity of water mechanically carried over with the steam, but it is useless from the stoker's point of view, as it gives no indication at the time when the waste is actually taking place. To ensure economical working it is essential that the air pass through the fire at a fixed rate, depending on the nature of the fuel and the thickness of the fire; thus, if, for a given fuel, this figure has been determined, it will merely be necessary for the stoker to regulate the damper accordingly, and this he can do if provided with an apparatus which will indicate continuously the quantity of air passing. The difference in pressure between the gases immediately above the fire and at a point near the damper must be a measure of the rate of flow; thus the apparatus needed resolves itself into a sensitive differential pressure-gauge. In a more perfect type of apparatus two gauges are used having their respective pointers mounted concentrically on the same dial; one of these shows the difference between the atmospheric pressure and that immediately above the fire, while the other indicates the difference between that at the last-named point and at the end of the flue near the damper. With this apparatus carefully calibrated, it is possible to tell from the indications on the dial the state of the fire as well as the conditions under which the combustion is taking place. For example, if at a certain time the difference between the atmospheric pressure and that at a point immediately above the fire was low, while that between the last-named point and one near the damper was high, the indication would be that the fire had become thin and more fuel was required. If then, after replenishing the fire the pointers should not return to their normal positions they must be made to do so either by cleaning the fire or by varying the opening of the damper, whichever course is indicated by the position of the respective pointers.

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[1] *See* De Ingenieur, 1908, p. 617.

[2] Bulletin de la Société de l'Industrie minérale, St. Etienne, 4e Série, Tome 9 1908, p. 569.

I. C. B.

Influence of Scale on Efficiency and Safety of Heating-Installations. E. Reutlinger.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin. 1910, vol. liv, pp. 636-42 and 676-81.)

The Author made numerous experiments on the influence of scale and other impurities on the rate of heat-transmission and on the physical properties of the metal composing the vessels in which the fluid was heated. The results are classed under four distinct headings:—

(1) The data obtained add considerably to the clear conception of the transmission of heat where the latter passes from a gas through

a plate to water, and is emitted by radiation, conduction and convection simultaneously. Values for the coefficient K are given for clean surfaces and for a range of temperatures of 572° to 1,292° F. It was found that the resistance to heat from plate to water is much greater than is generally supposed. For surfaces covered with a scale it was found that no definite rule expressing the relation between the resistance and the chemical composition of the scale could be formulated. The experiments further indicated the extent of diminution in the rate of transmission and the rise in temperature of the plate.

(2) The percentage-decrease in the value of the coefficient K for transmissions of heat from hot gases, steam, or fluids to water was determined.

(3) The Author calculated the loss which would accrue from a layer of dirt deposited on the heating-surfaces of any type of modern heating- or cooling-plant. He laid special stress on the difference existing between transmission from steam to water and from gas to water. In the former case a layer of dirt may suffice to halve the utilization of available heat, whereas in the latter this loss is only one-tenth of the whole. It is shown that too much importance is attached to the influence of scale on the transmission of heat in boilers, and not nearly sufficient attention paid to the detrimental effect of such impurities, in the case of other heating-surfaces, such as evaporating-pans, radiators and condensers. The calculations based on the formulas in Section 1 gave results which are in agreement with practice.

(4) The Author finally investigated the influence of deposits of oil and fat on the rate of transmission, on the elasticity of the material, and on its toughness. It was shown that for safety either soft water must be used or the boilers cleaned frequently.

C. J. G.

Improved Stroke-Reducing Devices for High-Speed Engine Indicators. K. KARGER.

(Zeitschrift des Oesterreichischen Ingenieur- und Architekten-Vereines, Vienna, 1910, vol. lxii, pp. 211-2.)

The Author describes three types of stroke-reducing devices for high-speed engine indicators. The first consists of two sliding pairs, and follows the motion of the engine-crosshead accurately at all parts of the stroke. This type is made by Hauber and consists of an indicator mechanism with wooden lath, which can be connected to the engine-crosshead by a movement releasing a spring, the lath being thereby pressed to the crosshead. It is immaterial at which part of the stroke the lath is released or thrown into gear. A further advantage of this system is that two separate indicators can be actuated by means of two cords connected to the common reducer. The elements of the mechanism are very light, so that its moment of inertia, even at high speeds, is small. The Author further describes the method of attaching these indicators to vertical and horizontal high-speed engines. This type of reducer is not suitable for long-stroke, high-speed engines, and two systems of reducing-pulleys for that purpose are described. The first consists of a double set of reducing-pulleys suitable for strokes of 24 to 80 inches. The other is of the worm and worm-wheel type, and was found both compact and efficient.

C. J. G.

Economical Production of Steam. Izart.

(Revue de Mécanique, Paris, 1910, vol. xxvi, pp. 223-57.)

This is the third of a series of articles on the subject and deals with the question of instruments for measuring draft. The Author is of opinion that the CO_2-recorder does not really fulfil the practical requirements of an instrument to assist the stoker, but that a recording-instrument for measuring draft is eminently suited for the purpose if applied to registering the difference of pressure across the bed of fuel. Several facsimile records taken with such instruments are given showing the results obtained by a good stoker and by a bad stoker, and exhibiting, moreover, the rate at which stoking is proceeding at any moment. Some thirty-five draft-gauges are described and illustrated, many of which are recording. They are divided into various categories, such as ordinary U-tubes, those using two liquids, differential recorders, etc.

H. R. S.

Cast-Iron Fittings for Superheated Steam.

Professor I. N. Hollis, E. F. Miller and A. S. Mann.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 841-72.)

In three Papers by the above Authors[1] many instances of success and of failure of cast-iron fittings were described. The general conclusions are summarized in Professor Hollis' reply and he points out that the differences of experience with this metal are due either to fundamental differences in chemical composition or to variations in temperature. Cast iron varies in its behaviour under high temperature, starting at about 450° F., and in many cases there are marked deteriorations in the structure and in the strength, which are much increased by fluctuating temperature. The final conclusion is that cast-iron fittings for superheated steam are inadmissible if the temperature is likely to fluctuate, but can be safely used when the temperature is constant.

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[1] Journal of the American Society of Mechanical Engineers, 1909, vol. xxxi, pp. 1361-80.

H. R. S.

Accuracy of High-Pressure Manometers. G. KLEIN.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 791-7.)

The necessity of careful measurements in connection with high-pressure tests has rendered the accuracy of high-pressure manometers of the utmost importance. After referring to spring manometers the Author briefly describes the Stückrath balance-manometer, the Marten differential manometer used for measuring small differences at high pressures, and manometers, such as those made by Krupp and Schäffer and Budenberg, consisting of an accurately-fitted cylinder and piston requiring no hydraulic packing, and by means of a viscous liquid reducing leakage to a minimum. The Author discusses fully the friction-effects in the latter instruments, and points out the difficulties of measuring the piston-area, owing to temperature-changes and alteration in form due to the high pressures. Numerous tests were made and tabulated, the results being shown by means of curves. The Author concludes that, for measuring pressures up to 15,000 lbs. per square inch. balance-manometers may be used giving a degree of accuracy of within 0·5 per cent.

C. J. G.

Theory of the Hydraulic Ram. H. LORENZ.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 88-90.)

After briefly describing the early history and the action of the hydraulic ram, the Author enters into the mathematics of the subject and arrives at the following results:—The theory of the hydraulic ram, as developed by him, enables the engineer to predetermine its mode of operation, and more especially the quantity of water required and the weight lifted, as well as the number of strokes per unit of time, from which the efficiency of the entire system may be readily calculated. The working of the ram depends mainly on the weight of the exhaust-valve, which may be adjusted between two extreme limits. The efficiency decreases from unity to zero as the load on the exhaust-valve increases from its lower to its upper limit. The amount of water required to work the ram, the quantity of water lifted per stroke, as well as the period of each stroke, increase with the load on the exhaust-valve. These results are recorded both in tabular and graphical form.

C. J. G.

Refrigerating-Machine using Water as the Working Fluid.

E. LEMAIRE.

(Le Génie Civil, Paris, 1910, vol. lvii, pp. 65-9.)

When any liquid is allowed to evaporate, heat is absorbed, thus, for refrigerating purposes, a constant supply of a liquid which can be made to evaporate at the temperature required is all that is necessary. In common practice the liquid used is either methyl chloride, ammonia or the dioxide of carbon or of sulphur; all these, at standard pressure, boil at a temperature well below the freezing-point of water. The vapour-tension of water, at low temperatures, is exceedingly small, and consequently water can only be used as a working fluid in the production of comparatively low temperatures, if the evaporation be allowed to take place in a more or less perfect vacuum. Hitherto, it has been found impossible in practice to maintain a sufficiently good vacuum, but the introduction of the steam-ejector pump wherein a powerful jet of water is driven through the nozzle, a type of pump brought out especially with a view to the requirements of the steam-turbine, has removed the difficulty, and water is now being used as the working-fluid in refrigerating-machines. The medium commonly used for the transfer of cold is brine, which, after circulating in the chambers to be cooled down, traverses a tank containing a nest of tubes in which the working fluid is at the time evaporating, where it is thus again cooled down. In the apparatus described, the brine, after having absorbed heat while circulating through the chambers, is introduced, in the form of fine spray, into an exhausted receiver where evaporation takes place, with the result that the brine is again cooled to the required temperature. The brine itself thus constitutes the working fluid and great simplification results therefrom. Three installations on this principle are described and illustrated. The first, at the Béthune mines (Pas-de-Calais), is employed principally for cooling the oil used for the extraction of benzol from the surplus gases of the coking-ovens, but is also coupled to a small ice-making plant; the second is fixed on board the French cruiser *Danton* for the purpose of lowering the temperature in the magazine and thus hindering the spontaneous decomposition which is not unknown to take place in the smokeless powders of the nitrocellulose series; and the third has been installed in a glue-factory at Nanterre (Seine).

I. C. B.

Multicellular Turbine-Machines. A. RATEAU.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 217-70.)

The Author confines himself to a descriptive survey of the subject, avoiding the theoretical side. Steam-, mixed and marine-turbines, centrifugal-pumps, turbine-pumps, turbine-blowers and turbine-compressors are treated. Under the head of action- and reaction-turbines practical details of diaphragms, distributors, wheels, shafts and vanes are given, and the Author discusses total and partial injection, output, number of wheels, and division of the fall in pressure, and gives reasons for his preference for action-turbines, and a description of the regulation by shutters of the two kinds of steam in mixed turbines. Marine-turbines comprise a large number of wheels with a low velocity of revolution. Details of their application on the *Voltigeur*, *Chasseur* and *Carabinier* are given. Centrifugal-pumps are usually coupled to an electric motor, while turbine-pumps are worked directly by a steam-turbine. Descriptions of the machines with drawings and characteristic curves are given. In conclusion, the Author points out that turbine-machines wear very little and need little lubrication, while the cost of maintenance is small.

W. H. B.

Turbine-Machines in Comparison with the Corresponding Piston-Machines. A. BARBEZAT.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 271-314.)

The subjects treated in this Paper are the Author's centrifugal pumps and steam-turbine, and turbine-compressors on the Rateau system. Details are given of two centrifugal pumps, one of 148 HP. capable of raising 88·3 cubic feet of water per minute to a height of 508·5 feet at a normal speed of 1,480 revolutions per minute, the other lifting 3,531·6 to 5,297·4 cubic feet of water per hour to a height of 721·8 feet at the same speed. For the former are given diagrams of the characteristics in terms of the output at various speeds, and a diagram of normal outputs and corresponding pressures, and the cases in which a centrifugal or a piston-pump is the more efficacious are also shown by a diagram. The Author gives a general description of the compression of the air at each element of the series in turbine-compressors (with details of cooling appliances), and of a multicellular compressor absorbing 325 HP. and capable of transmitting at 4,000 revolutions per minute 37·5 cubic feet of air per second at a pressure of 61·14 lbs. per square inch. Reference is made to a comparison between a Meyer piston and a Rateau turbine-compressor, and Rateau's system of regulation is described. The steam-turbines of Laval, Parsons, and of the Author are described, and the Paper concludes with a study of the dynamics of the regulating-apparatus of turbines.

W. H. B.

Remarks on Turbine-Machines. J. Rey.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 315–41.)

Steam-turbines, multicellular centrifugal pumps, multicellular compressors, and turbine-pumps are considered in this Paper. With regard to steam-turbines the Author treats of constructional details, different forms of vanes and packing, and describes the results of tests on a steam-turbine connected to a hydraulic brake capable of absorbing 1,479 HP. at 3,000 revolutions per minute; also a mixed turbine for a compressor and its system of regulation. He gives an account of centrifugal multicellular pumps since 1858 and general details of construction. Diagrams of height of lift and of output for motor and pump are given for pumps raising respectively 5,297 cubic feet per hour to a height of 229·6 feet, 1,412 cubic feet per hour to a height of 1,213·9 feet, 35,316 cubic feet per hour to a height of 29·5 feet, and 14,126 cubic feet per hour to a height of 88·5 feet. Details of velocities of revolution and of percentage output for the entire group and for the pump alone are given in each case. Under multicellular compressors a comparison of the old and new types and a description of a compressor of 1,282 HP. working at 3,000 revolutions per minute are given. The Paper concludes with a short note on a 493-HP. turbine-pump.

W. H. B.

Application of the Steam-Turbine to Locomotives. G. Belluzzo.

(Il Politecnico, Milan, 1910, pp. 161–6.)

The Author first discusses the special requirements of the steam-turbine for locomotive work as distinct from marine-propulsion. A turbine suitable for locomotives must be capable of developing its full power at various speeds or various powers at the same speed; of starting under load; and of developing equal power in either direction of rotation. The first requirement is met by allowing the steam to act on one or more blades on a single wheel according to the speed required. The variation of efficiency under different conditions of speed is of about the same order as that obtained under similar conditions with reciprocating-engines. The ability to run in either direction is obtained by having two rows of blades on each wheel with opposite curvature, the steam being admitted from opposite sides of the wheel according to the direction of rotation required. The loss of efficiency due to windage in the idle blades is not more than 2–3 per cent.

Experiments have been made with a small locomotive weighing 26 tons and working at a pressure of about 22 lbs. per square inch, equipped with four turbines (two high- and two low-pressure) of a total capacity of 100 HP. There are two pairs of driving-wheels and the turbines are arranged with the high-pressure casings on one side of the locomotive and the low-pressure on the other. The trials were made in a circular route about 1,600 yards in length and laid out with various curves and gradients in order to test fully the performance under all conditions. The results were very satisfactory and showed a steam-consumption of only 36 lbs. of steam per horse-power-hour whether in a forward or backward direction.

Some of the advantages of turbine-driving, in the Author's opinion, are:—(1) The possibility of increased velocity, (2) Less lubrification required, (3) Economy of coal-consumption, (4) Less maintenance and repairs.

L. F. M.

Test of a 1,000-*HP. Steam-Turbine, with Special Reference to the Shrouding and the Distribution of the Blades.*

Prof. E. Josse.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv pp. 121-8 and 167-70.)

The Sächsische Maschinenfabrik steam-turbine is described in detail, with special reference to the method of fixing the blades Tests on a turbine of 1,000 HP., carried out with two distinct types of rotors, of which the one was not provided with shrouding, whereas the other was shrouded and its blades spaced more closely together, showed the latter type of rotor to be more efficient. The advantageous effect of shrouding and of a judicious spacing of the blades in the high- and intermediate-pressure stages is deduced from curves, while it was impossible to find a means of increasing the efficiency of the low-pressure stage. In conclusion, the friction- and windage-tests are recorded, and the latter discussed with special reference to the steam-packing employed in the stuffing-box on the low-pressure end of the turbine. The turbine was found to work satisfactorily. The shrouding was reinforced with nickel-steel wire of 0·4 inch diameter and of 75 tons per square inch ultimate strength. At speeds up to 4,500 revolutions per minute, and with the shrouding removed, the wire showed no signs of breaking.

C. J. G.

Test of a 15,000-*Kilowatt Engine-Turbine Unit.*

H. G. Stott and R. J. S. Pigott.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 315-72.)

The well-known Manhattan engines have an economic rated load of 5,000 kilowatts, but can work at 8,000 kilowatts continuously, with a greater steam-consumption, however. Additional power was needed, and the question was debated whether to replace these engines by turbine-units or to apply exhaust-turbines to them. The matter is fully discussed and it is shown why the latter arrangement was adopted. The addition of a turbine-unit of the Curtis type to each of the Manhattan engines enables the combined set to develop 16,000 kilowatts. Elaborate tests have been made, some of the important results being as follows:—

There is an increase of 100 per cent. in the maximum capacity of the plant and an increase of 146 per cent. in its economic capacity; the over-all steam-economy is 13 per cent. better than the highest high-pressure turbine result, and there is an average economy of 25 per cent. over the results obtained with the original reciprocating-engine alone. The average thermal efficiency for loads between 6,500 and 15,500 kilowatts is 20·6 per cent. and the maximum power obtained in these tests is 16,172 kilowatts. The efficiency-ratio of the reciprocating-engine by itself was obtained, and for loads between 4,500 and 8,000 kilowatts was 72 to 76 per cent. The efficiency-ratio of the turbine by itself was also ascertained, and between 2,500 and 8,000 kilowatts (developed by the turbine alone) was nearly constant at 60 per cent. The efficiency-ratio of the combined reciprocating-engine and turbine was 68 per cent. at 7,000 kilowatts, 71 per cent. at 10,000, and 65 per cent. at 16,000 kilowatts.

H. R. S.

Efficiency Test of a Schulz Turbine. M. F. GUTERMUTH.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 82-8.)

The Schulz turbine, like the Curtis turbine, is of the axial-flow impulse-type divided into pressure-stages and subdivided into velocity-stages. It differs from the latter in that it can be governed at each stage by altering the number of active admission-nozzles according to the amount of steam required, whereas the Curtis turbine can only be governed at the first pressure-stage. In order to effect this intermediate regulation of the steam-supply the nozzles of the second and succeeding rotor-wheels can be opened or closed by means of annular slide-valves. The turbine tested is of 650 HP. normally, but can develop a maximum of 900 HP. Tests were carried out (1) with intermediate regulation, in which all the nozzles were adjusted according to the amount of steam required for each stage; (2) without intermediate regulation, viz., only the first set of nozzles were operated, the intermediate nozzles remaining fully opened throughout; (3) for comparison with ordinary impulse-turbines steam was throttled before being admitted to the first set of nozzles, which latter were kept fully opened. The turbine was also tested at very low speeds, both with and without intermediate regulation, to study its behaviour when running under marine conditions. The results of the tests are tabulated and represented by means of curves.

The Author concludes from his experiments that intermediate regulation of the pressure-stages in an impulse-turbine must undoubtedly be regarded as an important improvement. In both tests, viz., running the turbine coupled to a dynamo and under marine conditions, it was fully shown that a notable economy in the steam-consumption and heat dissipated can be obtained by intermediate regulation. This advantage increases as the load is decreased, while regulation is easily effected either by hand (for marine turbines) or automatically (for electrical work).

C. J. G.

Comparative Tests of Zoelly Steam-Turbines.

(Schweizerische Bauzeitung, Zürich, 1910, vol. 55, p. 108.)

The firm of Escher Wyss and Co. of Zürich have built a number of Zoelly steam-turbines and made comparative tests of steam-consumption and efficiency. The chief results are given in the form of a Table. Each turbine was direct-coupled to its dynamo, and the dynamos were of 300, 700, 1,000 and 2,300 kilowatts output respectively. The largest size was built to run at 1,500 revolutions per minute and the rest at 3,000 revolutions per minute.

The results of several tests for each size of turbine are given, the following figures being the lowest for each machine :—

Nominal size	300 Kilowatts.	700 Kilowatts.	1,000 Kilowatts.	2,300 Kilowatts.
Test-output—Kilowatts	315	859	1,060	2,507·7
Speed—Revolutions per minute	3,030	3,000	2,999	1,501
Steam data—				
Inlet-pressure—Lbs. per sq. inch.	112·5	193·5	153·0	187·2
Temperature—Fahrenheit	626·9	579·2	572·0	607·1
Exhaust-pipe vacuum [1]—Per cent.	96·6	94·8	95·3	93·3
Consumption—Lbs. per kilowatt-hour	16·1	14·8	14·6	14·08
Do. per turbine-HP.	10·8	10·14	9·8	9·85

E. R. D.

Test of a Jaeger Turbo-Blower. H. Mitter.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 218-27.)

Very few experiments have been made with rotary or turbo-blowers and the Author, therefore, published the results of exhaustive tests of one built by C. H. Jaeger and Company. The blower is driven by a 50-HP. Siemens-Schuckert motor, and at full load runs at 2,935 revolutions per minute, delivering 3,550 cubic feet of gas per minute at a pressure of 19·5 lbs. per square inch. The gas used had a density of 1·15 relatively to air. The air was drawn in through a short length of piping provided with a regulator-valve, and blown out through a nozzle attached to the end of the pressure-piping. The tests were each of about 1 hour's duration, and during this period temperature- and power-readings were taken regularly. The velocity of the issuing air-jet was obtained by Weisbach's formula and from the temperature and pressure of the air before and after the nozzle :—

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[1] Barometer reading not stated.

$$w_0 = \phi \sqrt{2g\, C_p \frac{1}{A} (273 + t_5) \left[1 - \left(\frac{p_1}{p_5 + p_1} \right)^{0 \cdot 29} \right]}$$

Where w_0 = air-velocity in metres per second; g = acceleration due to gravity; 9·81 m/sec^2; $C_p = 0 \cdot 237$—specific heat at constant pressure; A = 1 : 425, the mechanical equivalent of heat; t_5 = temperature before nozzle in degrees Centigrade; p_5 = the gauge-pressure before nozzle in millimetres of water; p_1 = the height of the barometer in millimetres of water; $\phi = 0 \cdot 98$ the coefficient of discharge of the nozzle.

From the velocity the weight of air displaced per minute can readily be obtained. The friction-losses were found to be only 2 per cent., which might be regarded as a favourably small fraction of the total output. The efficiency at 50 per cent. overload was 55 per cent., at normal load 76 per cent., and at half load 73 per cent. These efficiencies are better than those obtained with a reciprocating-blower.[1]

C. J. G.

Air-Filters for Turbo-Dynamos. K. Hoefer.

(Zeitschrift für das Gesamte Turbinenwesen, Munich, 1910, vol. vii, pp. 225–9 and 250–2.)

It has been found desirable to remove dust from the air used for cooling turbo-dynamos. Cyclones are not satisfactory because they only remove the coarser particles, and cleaning by water-spray is costly and has the disadvantage that water is carried over in a fine state of division and tends to injure the insulation. The more satisfactory method is to draw the air through some filtering material, and a special cloth made of cotton-wool has been found to give good results. This cotton-wool cloth can be mounted in separate pieces on frames which can be slipped into a case, and are easily drawn out for cleaning purposes, or the material can be in one continuous length stretched over a number of hinged frames, which can be closed up for working and pulled out, like an accordion, for cleansing purposes. Designs and photo-reproductions of such filters are given, as well as two methods of application. Although the introduction of these filters is only recent, it is stated that over a hundred are already at work. The filter-cloths require cleaning every 2 to 8 weeks, according to circumstances.

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[1] *Zeitschrift für das Gesamte Turbinenwesen*, 1908, p. 171.

H. R. S.

18,000-*HP. Water-Turbine.* H. B. TAYLOR.

(American Machinist, New York, 1910, vol. xxxiii, pp. 289-92.)

Four of these turbines, said to be the largest in existence, have recently been installed at Oroville, Cal. The shaft is vertical and drives an electric generator of 10,000 kilowatts capacity. There is a single runner of the inward-flow Francis type; the head is 525 feet and the power developed at 400 revolutions per minute is 18,000 HP. Hitherto such reaction-turbines have not been applied for heads over 300 feet. The runner is constructed of a special bronze and revolves in a cast-steel volute casing. The guide-blades are made of cast steel, and are movable so as to regulate the amount of water delivered in proportion to the load. Each guide-blade has a shank passing through a stuffing-box, which is then connected by means of a lever and a link to a cast-steel operating-ring. The position of this ring is regulated by means of two pistons hydraulically-controlled by an electric governor; the ring begins to move for a variation of ½ per cent. of speed. The hydraulic thrust on the runner is balanced by admitting water under pressure to its underside, and the weight of the runner and of the rotor of the generator is taken by an oil thrust-bearing. This bearing consists of an upper disk fixed to the shaft and of a lower disk supported on the foundations; between the two disks there is an annular chamber into which oil is forced under a pressure of 250 lbs. per square inch. Great care was exercised is designing the guide-vanes and runner-vanes to prevent erosion and pitting, and with success, the first unit put into operation in December, 1908, having so far shown no signs of erosion on the vane-surfaces. Careful tests of efficiency were made, the head and quantity of water being measured by means of Pitot tubes. The following results were obtained for the over-all efficiencies.:—No. 1 unit, 87 per cent. at 87 per cent. of the full load; No. 2, 87 per cent. at 90 per cent. of full load; No. 3, 88·2 per cent. at 93 per cent. of full load; and No. 4, 89 per cent. at 91 per cent. of full load.

H. R. S.

Mine Experimenting-Station at Liévin. J. TAFFANEL.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910,
vol. i, pp. 166-83.)

Since 1891 France has had only one great accident due to fire-damp, but unfortunately the strenuous measures taken to ensure a rapid circulation of air in mines have brought about the danger of firing in the coal-dust stirred up. The experimenting-station at Liévin was founded by the Central Committee of the Collieries of France to enquire into all questions connected with the safety of mines and especially into the firing of coal-dust. It contains two galleries, one for experiments with safety explosives and one with coal-dust; tests on the explosion of fire-damp are also carried out, and the Author describes the methods and apparatus employed and discusses the initial causes of firing. Dynamite should be abandoned in favour of safety explosives, whose flame is shorter and colder. The inflammability of coal-dusts depends on their fineness, dryness and richness in volatile material. Various tests are described, and methods to check the progress of explosions discussed, amongst others deadening arrangements by the interposition of incombustible material and watering.

W. H. B.

Federal Investigation of Mine Accidents, Structural Materials and Fuels at the United States Testing-Station, Pittsburg, Pa.

H. M. WILSON.

(Proceedings of the American Society of Civil Engineers, New York, 1910,
vol. xxxvi, pp. 360-468.)

The mine accident which occurred at Cherry, Ill., on the 13th November, 1909, when 310 men were killed, has called attention to means for preventing such disasters. The Author gives an account of the experiments of the United States Geological Survey since 1904 on structural materials, such as stone, clay, cement, etc., and tests and analyses of coals, lignites and other mineral-fuel substances. The average death-rate in the coal-mines of the United States is three times that of France, Belgium and Germany. The mine-rescue investigations carried on at Pittsburg include:—(1) The relative explosiveness of mine-gas and coal-dust; (2) Tests to determine the relative safety of various explosives; (3) Safety of electric equipment; (4) Safety of mine-lights; (5) Tests of breathing-apparatus. As the result of many tests, a list of permissible explosives was issued. In connection with the use of electricity in mines, a complete series of tests was made on the power of enclosed electric fuses to ignite a gaseous mixture. The Author describes the method of testing lamps and also mine-rescue methods. For the purpose of testing explosions of gas and dust, a gallery is employed 100 feet long, with a minimum internal diameter of 6⅓ feet, and the explosive is fixed in a cannon. It was found that coal-dust taken from the roads in one of the coal-mines in the Pittsburg district required at least 12 per cent. of water to prevent ignition. It has also been proved that the finer the dust the more water is required. The disruptive force of explosives is measured by the amount of oscillation of a ballistic pendulum, which consists of a 12-inch mortar weighing 14·1 tons. A steel cannon is mounted on a truck set on a track laid in line with the direction of the swing of the mortar. The Author describes various pieces of apparatus, such as calorimeters, impact-machines, apparatus for measuring rate of detonation, etc. For investigating structural materials a large amount of apparatus is at the disposal of the department.

R. S. B.

Improved Method of Granulating Blast-Furnace Slag.

G. Jentsen.

(Stahl und Eisen, Düsseldorf, 1910, pp. 824-7.)

Slag-sand produced by the cold water-jet process is known to possess valuable hydraulic properties, which make it very suitable for use with cement. But its high content in water, 30 to 40 per cent., makes artificial drying necessary, which is both difficult and costly by reason of the porous and spongy nature of the material. To avoid this objection to the water process without losing the hydraulic properties of the slag, other methods of granulation have been proposed, such as the steam-jet, the compressed-air jet, and running the molten stream upon some rapidly revolving disk or wheel. In all cases, the granulated material was made to fall upon some form of fixed iron plate. But though means were adopted to cool the plate, a tendency of the slag to stick and to agglomerate into lumps constituted a serious practical difficulty.

The Author describes a novel arrangement of plant, designed to avoid this difficulty. The slag flowing from the furnace is led into the higher end of an inclined revolving iron cylinder some 6 feet in diameter and 50 feet long. Below the inflowing stream is set a flat and broad nozzle from which a jet of compressed-air is directed into the fluid mass. The latter is thereby broken up and thrown, in granulated form, against the walls of the cylinder, which is very effectively cooled by water-sprinkling on the outside. The revolution of the cylinder brings constantly fresh portions of its surface into contact with the hot material. As the latter is kept moving, and its distance of travel is considerable, it does not stick to the iron, nor do the grains agglomerate; but practically the whole of the charge arrives at the lower end of the cylinder in the form of sand, the few larger pieces being no bigger than a cherry-stone. This sand falls from the lower end of the cylinder directly on to an elevator which lifts and discharges it into a raised storage-tank. From this tank it may be shot, as required, into a railway-truck run underneath. A ventilating shaft over the lower end of the cylinder and the bottom portion of the elevator carries off the steam and noxious fumes generated. An arrangement for moistening the compressed-air prevents the formation of slag-wool. This plant has been in successful operation more than a year, giving a daily output of 100 tons of slag-sand. It is intended to apply the system to all the furnaces at the works, and to that end a special turbo-blower is to be erected, the air-supply absorbing about 10 HP. per furnace. The cylinder and elevator together require 8 to 9 HP.

The advantages of granulating by means of compressed-air are numerous. While giving an improved material, a notable saving of labour and cost is effected. Not the least of these advantages is the smaller volume of the air-granulated material, a cubic metre of which weighs, as an average, 1,300 kilograms against 520 kilograms for the water-made sand. Besides its special value as a substitute for river-sand for cement—for the hydraulic properties of the water-granulated material are retained—it may be expected to come into great demand for road-making and as a cheap and convenient packing for mines. G. G. A.

Blast-Furnace Slag for Concrete. Dr. H. Passow.

(Stahl und Eisen, Düsseldorf, 1910, pp. 829–36.)

The Author gives a detailed account, with tabulated results, of a series of experiments and trials made with concrete containing a large proportion of blast-furnace slag. Test-blocks were subjected to a crushing pressure after 1 month, 3 months and 6 months of hardening. The results obtained are generally favourable to the use of slag for this purpose; but they tend to overthrow certain prevalent beliefs concerning the unsuitability of some forms and qualities of slag as a constituent of concrete. When containing slag in pebble- or gravel-size pieces, properly made concrete is shown to be satisfactory in every way and suitable for all kinds of work where such a compound is required. Also these experiments make it appear that certain qualities of slag which disintegrate slowly when exposed to atmospheric influences do not crumble when forming an ingredient in concrete. As iron rusts when exposed to the air and keeps bright when imbedded in cement, so slag which disintegrates in the open is not acted upon by the air when encased with cement. Thus the objection to the use of such qualities based on its lack of stability in air falls to the ground. Concrete containing it proved to be in no respect inferior to that made of other qualities. Hence it appears that the quality of the slag used for coarse concretes is of less importance than is generally believed. When, however, these crumbling qualities are reduced to the form of sand and used alone with Portland cement in lieu of river-sand, the blocks proved to be much inferior in resistance to crushing to those made of other qualities of slag, a result due, in the Author's opinion, to the chemical rather than to the physical properties of the material. Another noteworthy fact learned from these trials is that equally good concrete was made from slag delivered from the blast-furnace works as "very suitable" and as "unsuitable" for the purpose. Evidently further trials are needed to ascertain whether the defects in the concrete expected are due to the slag or to some other cause. These experiments confirm the general opinion that slag-sand made by the compressed-air process is in a marked degree superior for use with cement to that produced by the water-granulating process. The Author affirms that the strongest cement does not necessarily make the strongest concrete; for he has found that in certain conditions an exceptionally strong cement makes a concrete inferior to that produced by a weaker quality. The results of these trials are given in twelve Tables.

G. G. A.

Reinforced Concrete for Mining Construction. VIEBIG.

(Glückauf, Essen, 1910, pp. 872-901.)

A remarkable fact in connection with mine engineering is the rapidly growing use of reinforced concrete, not only for surface erections, but in underground constructions, particularly for the lining of the shaft, the pit-eye, the roadways and important cross-cuts. For such uses, and all kinds of tunnelling, this new combination of materials has proved specially adapted. Pure concrete offers great resistance to compression; but it is not a material to be subjected to transverse or tension strain. This defect, however, which has hitherto restricted its use, has been to a large extent removed by embedding in it iron of convenient form and in the required position. Thus, in the case of an arch heavily loaded on the crown, tensile strength is required in the lower half of the thickness of the ring. The ability to give by this means some degree of elasticity as well as tensile strength to concrete has made of it a suitable material, where, as in mines, side-thrusts and pressures coming from unknown directions have to be provided for. The watertight nature of the concrete, too, is a quality of great practical value in a material used for shaft- and tunnel-lining and dam-construction, and this quality is made available under nearly all conditions by reinforcement. The large number of mine-shafts in Germany lined or being lined wholly or in part with reinforced concrete is evidence of an already wide recognition of its value. Recent experience in the United States has shown it to be particularly well adapted for sinkings through water-bearing strata. A minor advantage of concrete over masonry, but one of considerable practical importance where there is a rock backing, is that the former prevents trickling of water between it and the rock.

. At one of the Gelsenkirch mines the abutments of an iron dam are made of reinforced concrete. The cast-iron segments were calculated to resist a pressure of about 1,570 lbs. to the square inch on the concrete where they abut. To withstand this strain the concrete used at those parts was made of 1 Portland cement to 1½ gravel. For the rest of the abutments the proportions were 1 to 3. By reinforcing, the strains were advantageously distributed. The fire-resisting qualities of concrete make it very suitable, in the reinforced form, for damming-off a mine-fire. A dam in which round iron rods of ⅜- and ½-inch diameter were used for reinforcing was built to shut off a fire. Though in a few days the dam became very hot, no cracks appeared. Hence it is to be recommended for explosives stores, stables, etc. For airways it offers little resistance to the ventilating current. Compared with walling it occupies less space and generally costs less. The applications of reinforced concrete in underground construction are numerous. At the Gelsenkirch mines last year 2,350 tons of cement and 540 tons of iron were used in that way. The Author, whose article is fully illustrated, deals exhaustively with the use of reinforced concrete in all kinds of mining construction.

G. G. A.

Economical Use of Compressed-Air. H. Reiser.

(Glückauf, Essen, 1910, pp. 679-85.)

While the economical advantages of high-pressure steam are universally recognized, for compressed-air a pressure of 90 lbs. is rarely exceeded when applied to mining operations. One consequence of this is that when power is required at points far distant from the mine-shaft new centres of production have to be made by installing small, separate, electrically-driven compressors in different parts of the mine. The Author believes that when the extra cost is taken account of it will be found that air can be conveyed to those points more cheaply if a higher initial pressure be used in conjunction with a higher temperature. His contention is that to use high-pressure air advantageously, and to avoid trouble with ice at the exhaust-ports, as well as to comply with the fundamental principle which requires compressed-air to be produced as cold as possible and to be used as warm as possible, some degree of superheating is necessary. This should be effected in suitable coal-burning superheaters near the pit-bank. The temperature should be raised 100° C. to 230° C. Under such conditions the conduit-pipes must, for the most part, be covered. It can be shown that raising the temperature diminishes the loss of pressure. By a suitable proportioning of the diameter of the pipes and the pressure, the loss in the air-conduits may be kept within practicable limits, and the multiplication of centres thereby avoided. The enhanced cost and loss of efficiency occasioned by increasing the diameter of the cylinders to compensate the loss of pressure in a long conduit-pipe is not always sufficiently considered. A better remedy is to be found in increased pressure and higher air-temperature.

In mines compressed-air is used chiefly for hauling, separate ventilating-fans and rock-boring machines. For the latter, in which simplicity of construction and action is desirable, the air cannot, with advantage, be used expansively. But for the other purposes there is, in the Author's opinion, no valid reason other than cheap materials and inferior workmanship. So long as the rubbing-surfaces pertaining to the slide-valve are not, in these mine-engines, made of good, hardened, Siemens-Martin steel instead of cast iron, so long will the non-expansion engine hold the field. Yet the economy of the expansion-engine in air-consumption is great. The Author affirms in conclusion that by the adoption of superheating and the use of expansion the cost of compressed-air may, in many cases, be reduced to one-half.

G. G. A.

Further Experience with Electricity in Mines.

R. NEIL WILLIAMS.

(Engineering and Mining Journal, New York, 1910, vol. xc, pp. 28-31.)

The advantages offered by electricity as a motive power in mines are chiefly economy and convenience. The Author discusses these two points in relation to the Davis Coal and Coke Company's mines in West Virginia. The economic advantage of electric power is there made evident by the pay-roll, for it has been proved that one locomotive can do the work of fifteen horses on an average. This means the employment of one good man in place of fifteen boys, and the expenditure of 10*s.* to 12*s.* for power, instead of 30*s.* for feed. The Company have systematically worked up to a well-thought-out plan of electrical distribution. Both alternating and direct currents are used in accordance with the conditions under which they are applied. For long-distance transmission alternating current is the more economical, but it is also the more flexible, being readily adapted to any distance by means of the simple transformer. Moreover the induction-motor is admirably suited for use in coal-mines, particularly for driving fans and pumps which run continuously. Requiring no brushes, or like devices, the rotor revolves very freely, with no friction other than that of the bearings. This form of motor ensures absolutely no sparking and requires little attention. But for haulage and hoisting purposes the Company has been led to adopt the direct-current series-motor, on account of its characteristic of maximum torque at starting. The distances being great a potential of 600 volts was adopted. Danger from this high voltage is said to be not great, inasmuch as the current is turned off while the shifts are changing. The system has been carried out consistently throughout the various workings, with 600 volts direct current for haulage and three-phase alternating current, at a frequency of 60 cycles, for all other purposes. The policy of buying uniform apparatus for all the mines, even to standardizing the make of machinery, has resulted in the almost entire absence of an electrical scrap-heap. The importance of the work done by induction-motors in ventilating and pumping is so great that these motors must be amply large for the work, a necessity which leads to a low power-factor for the whole system. It has, therefore, been found desirable to bring up the general power-factor as near as possible to 100 per cent. by means of units independent of the generators. Rotary condensers, or synchronous motors, are suitable for this purpose, whether running idle or with load; but it is not always possible to provide a suitable load for a synchronous motor in the interior of the mine. This type of machine requires more attention than an induction-motor and is more susceptible to fluctuations in the supply of electric energy.

G. G. A.

"*Safety*" *Explosives with Alkaline Salts.* TAFFANEL.

(Annales des Mines, Paris, 1909, vol. xvi, pp. 512-54.)

The Author deals with the latest researches in explosive compounds of the so-called "safety class"; i.e., compounds which may be used in fiery coal-mines with the minimum of risk of igniting the gas. Mr. Dautriche some time ago made known his discovery that the presence of alkaline salts in explosives capable of yielding combustible gases hinders the combustion of those gases, and thereby reduces the flame of explosion, and that the addition, in small proportion of such salts to the safety explosive, appears to lessen the risk of ignition. These results were considered by the Commission of Scientific Research on Firedamp so important from the point of view of the safety of miners that, in accordance with its instructions, a complete set of experiments was carried out at the testing-station at Liéven to ascertain with precision what the effects of an addition of these salts are. The flame-producing substances experimented with were gun-cotton and trinitrotoluene, the former pure and with 2 per cent. of carbonate of soda, or 3 per cent. of sulphate of potash; the latter pure and with 5 per cent. of nitrate of potash or 10 per cent. of nitrate of barium. The flames from the shots were photographed and afterwards measured. The results in every instance showed a notable reduction in the length and intensity of the flame when the alkaline salts were added. A second series of experiments was then undertaken to ascertain the effect of these salts when added to the ordinary safety explosives, composed chiefly of ammonium nitrate and nitrotoluene, when fired in a sensitive mixture of gas and air. Again the results confirmed Mr. Dautriche's statements, the best effects being observed with nitrate of potash in the proportion of 5 per cent. The addition of this salt made it possible to double, in some instances, the weight of explosive without increasing the risk of ignition. A third set of experiments with coal-dust in the place of gas, gave disappointing results. For in a dust-laden atmosphere the addition of the alkaline salts appeared to increase the risk of ignition, thereby showing that the conditions of ignition are different in the two cases. Thus the practical lesson of these official trials is that the addition of 5 per cent. of saltpetre to the other components of a safety explosive lessens the risk of igniting the gas in a notable degree; but that the benefit of this moderating substance is to be obtained only in places free from accumulations of coal-dust. This leaves the dust problem unsolved.

G. G. A.

New Method of Packing in Mines. ROSENBECK.

(Glückauf, Essen, 1910, pp. 565–9.)

Practical trials have been carried on at the Katherina Colliery, in the South Essen mining district, with a view to utilize the refuse from coal-washing as a material for packing underground. With this system of gob-packing the two processes are related, inasmuch as water is required to give a certain fluid-consistency to the packing material. Essential and distinguishing features, from which the system derives its greatest advantages, are the small proportion used and the novel method of dealing with it. The refuse, i.e., the material of the wash-heap, is dropped down the shaft through a 10-inch pipe to the level where it is required, and conveyed thence, in its relatively dry state, by horse-haulage in tip-trucks of half a cubic metre capacity, to the places where it is to be deposited. But before discharging the contents of the truck, water, led from behind the shaft-tubbing, is added to reduce the mass to the consistency of a thick mud, the specific gravity of which averages 1·85. When this semi-fluid mass is tipped from the truck behind the working places, the faces of which are being worked to the rise, it spreads out and flows slowly back into the worked-out space which it is intended to fill from floor to roof. Obviously the system is applicable only to highly inclined seams. The minimum dip required is said to be 40°. The quantity of water necessary to give only just sufficient fluidity to the mass of packing materials is dependent on the dip. With a fall of 72° 20 per cent. is sufficient; while with a 55° slope 35 per cent. is required.

Where the conditions render this system applicable the results are said to be very satisfactory. It has been found that 0·84 cubic yard of normally dry refuse fills, when wetted with 35 per cent. of water, 1 cubic yard of the worked-out space, so that at most the sinking cannot be greater than 16 per cent. of that space. This is a better result than it is possible to obtain by pipe or launder discharge of a more fluid mass. The total cost of this method of packing at the Katherina Colliery is about 7½*d.* per ton of coal extracted.

G. G. A.

Hydraulic Packing for Mines. M. A. Clapier.

(Bulletin de la Société de l'Industrie Minérale, Saint Etienne, 1910, vol. xii, pp. 71–105.)

During the last ten years mining engineers have given much attention to the packing of the worked-out parts of a mine with a view to substitute mechanical for hand labour. Considerable progress has been made in this direction by the adoption of water as the carrying agent, packing material being conveyed to the empty spaces behind the working-faces through pipes in most cases, and sometimes, though rarely, by means of launders. The simplicity of this method, its saving of cost, and the entirely satisfactory results obtained, 90 per cent. of the spaces being filled, have led to its introduction into many collieries where the coal-seams are of only moderate thickness. The Author points out methods of working thick seams which favour the use of the hydraulic system and allow of it being further simplified and cheapened by the use of launders in place of pipes. A determining point is that the inclination of the launders for distributing the packing materials must be sufficient to prevent choking up. When screened and not very coarse materials are used, the water will keep them moving forward when the launders are inclined at an angle of 15°. The presence of some larger pieces will require more water. With an inclination of 25° the flow of material is easy and rapid, even with a reduced quantity of water. A desirable inclination is 20° to 25°, for then a small quantity of water is sufficient to carry unscreened or coarsely-screened materials. Where the conditions allow a slope of only 15° and under, fine materials must be used. These are important factors in determining the method of working the seams when it is desired to adopt the hydraulic system. In thick seams of less than 25° dip, the workings may be to the rise, the packing following the working-face up the rise. In thick seams lying at a higher angle, the Author recommends the cut, i.e., the slope of the working-face, to be intermediate between the horizontal and the dip of the seam, so as to give a suitable inclination to the launders, and he describes a method by which panels may be worked in that way. Illustrations show the general plan of a hydraulic gob-packing system, with arrangement of conduit-pipes and distributing-launders. The system, however applied, obviously tends to give security from coal-dust explosions.

G. G. A.

Improvements in Rock-Boring Machines.

(Zeitschrift für das Berg-, Hütten- und Salinenwesen, Berlin, 1910, pp. 83–5.)

The hammer form of machine rock-drill has recently been freed from some of the defects which have delayed its general adoption. Trials of new designs of this type were carried out in 1909 at the Maybach mine in the Saarbrücken district. Excellent results were obtained with one form in which compressed-air is used expansively and consequently economically. An incidental advantage of such a design is a less violent back-thrust. While the men working the common form of machine were unable to hold the drill up to its work longer than 5 minutes without rest, those who had charge of the improved form, known as the "Westphalian Bohrhammer," could go on for twice that time. Moreover, it was found that in consequence of the better construction, the time occupied in changing a bit was only 5 seconds for the Westphalian machine against 60 to 70 seconds for the ordinary forms. These trials included some experimental runs in sandstone and conglomerate to test the relative durability of electro-steel and steel of the commonly used quality for the borer bits. The results showed an average duration of 180 minutes for electro against 109 minutes for the ordinary quality, the time required to wear down the cutting-edge. The electro-steel showed, however, a greater tendency to splinter; but the remedy for this is probably to be found in the tempering.

The severe strain on the muscles of the arms when holding a hammer-drill up to its work in overhead boring often necessitates the employment of two men, the usual method being to pass a stout stick or bar through the looped handle of the machine to enable each man to contribute his share to its support. As they must stand under the machine, they suffer a good deal from the débris falling from the bore-hole. These disadvantages have been removed and the efficiency of this class of machine largely increased by the adoption of a pneumatic bar, introduced by G. Schol of Essen, and here described and illustrated. Already in use in several of the Westphalian mines, it may be said to have proved its value in practice. For use with the pneumatic bar the machine has no loop-handle; but in place of that means for obtaining a hand-grip on it, there is a rod sliding as a piston in a truly-bored tube serving at once as an air-cylinder for this piston and as a support for the machine. When set in position, compressed-air is led into it through a branch from the flexible supply-tube, and the drill is forced up and firmly held against the rock by air-pressure.

G. G. A.

Large Core-Drilling.

(Engineering and Mining Journal, New York, 1910, vol. lxxxix, pp. 1156-7.)

At the Stella Mine, Hermon, N.Y., 20-inch bore-holes are now being sunk through various parts of the pyrites ore-body in place of winzes and raises. These holes are to be enlarged by carrying the stopes up alongside. Three such holes, each about 50 feet deep, have already been bored. This depth was reached in eight shifts, the greatest depth bored in one shift being 13 feet. The longest piece of core removed was 6 feet. Machines are built to drill 30-inch holes, suitable for ore-shutes. The cutting-power of these drills is remarkable. A Terry 30-inch core-machine used in work for the U.S. War Department was recently applied to bore through a trap-rock concrete embankment. In some parts the concrete was reinforced with steel girders, and in others with iron drain-pipes. In one of the cores, shown in an illustration, there is embedded a section of one of these 8-inch I-girders, and in another a portion of a 2-inch iron pipe. The cutting is performed by "shot-bits." A groove on the inside of the bit serves as a passage between the rock-core and the wall of the bit, down which chilled-steel shot can travel freely. The quantity of shot fed to the bit must be enough to cover the bottom of the annular passage it is cutting. At the bottom of the bit an opening is cut for a water-way. The actual cutting is done by the breaking of the shot embedded in the softer material of which the bit is formed. This action produces hundreds of minute cavities in which the small particles of shot are constantly being caught, each time presenting a new facet and grinding away the rock. The action continues until the shot-particles become so small that the flow of water forces them out of the bore-hole above the drilling-tool. The peripheral speed of the bit is for small cores up to 5 inches diameter about 350 revolutions a minute, and proportionally less for larger diameters. The wear on the bit in the trap-rock reinforced concrete was about ¼ inch per foot cut and 10 lbs. of shot were used for that length of cut. The cost for 14 feet of that 30-inch boring was 21*s*. for wear of bit and 8*s*. for shot, oil, grease and waste. For the 20-inch holes at the Stella mine the cost of the wear on the bit is said to be only 9½*d*. per foot bored. The total cost of drilling a 10-inch artesian well, 2,000 feet deep, in New York City, where labour is dear, amounted to 22*s*. per foot. The large-core drill is recommended for the improvement of the ventilation in coal-mines, and in some circumstances for rescue-work. An illustration is given of the Terry 30-inch drill.

G. G. A.

Improvements in Shaft-Sinking Plant. E. LEROYER.

(Bulletin de la Société de l'Industrie Minérale, Saint-Etienne, 1910, pp. 283-303.)

The Société des Mines de Dourges has recently been engaged in shaft-sinking operations which have afforded opportunity for introducing improvements in the plant employed, particularly in the winding-plant. The Author describes a steam windlass designed to meet more fully the requirements of this kind of work. But the greatest advance is shown in the construction of an electric windlass, or winding-engine, adapted to the conditions of shaft-sinking. This windlass is driven by a three-phase motor, running with a current of 240 volts and 50 periods. It has two drums, each about 4 feet 7 inches in diameter and 1 foot wide. The motor is connected with the drum-shaft through a double train of gearing, so calculated that the normal speed of 585 revolutions of the motor corresponds to a speed of about 11 feet per second in the shaft. A hand-brake acting on the motor-shaft by a counter-weight, its normal position being 'on," is controlled by means of a pedal within reach of the man in charge. It is so connected with the governing-lever that it cannot be applied except when the current is off, and the motor cannot be started unless the brake is off. A second safety hand-brake acts upon a pulley fixed on the drum-shaft. The grip of this brake is also given by a counter-weight. Application of the brake cuts off the current by means of an interrupter connected with the governing-lever. This brake comes into action (1) through the movement of a pedal by the man in charge; (2) when the electric current ceases; (3) when the indicator points to a certain position of the cage or kibble; and (4) through a contact-breaker on the headgear acting to prevent over-winding. The position-indicator can be made to show the position of the kibble at any depth in the shaft. The resistances are calculated for about twenty-five windings an hour without heating, and to allow of an exact regulating of the speed for inspecting the shaft. All the parts of the electric controlling-apparatus are fixed to a cast-iron column and are well in sight. A load is lowered a short distance by means of the brake and a counter-current. But when the depth is sufficient use is made of the property of three-phase asynchronous motors to limit automatically the speed of their rotor to that corresponding to the synchronism. The descent is then made with the current till the bottom is nearly reached, when the reverse current and brake are brought into requisition.

The head-gear or pit-head frame, which is 40 feet high, carries two pulleys of 5 feet 3 inches diameter, the winding-ropes being of steel wire, 1 inch in diameter, and of the "anti-gyratory" type. There is a third and central pulley, over which is suspended the platform for the workmen when putting in the shaft-lining. The kibbles are of sheet-iron, barrel-shaped, 3 feet in diameter at the middle and 4 feet in height, giving a capacity of about 122 gallons.

G. G. A.

Circular v. *Rectangular Shaft-Sinking.* HENRY M. PAYNE.

(Engineering and Mining Journal, New York, 1910, vol. lxxxix, pp. 231-2.)

Coal-mines in America, like those in Scotland, have for the most part rectangular shafts, while English, French, Belgian and German collieries have, in nearly all cases, shafts of circular form. With a view to ascertain whether the distinction is based on sound reasoning or is due merely to custom, the Author visited all the above-mentioned countries. The results of his inquiries are given in brief. In the matter of rapidity of sinking he finds nothing to choose between the two forms, if the time be taken from start to completion. The same may be said of the total cost. But in the matter of upkeep the circular form has a marked advantage, for the lining of such a shaft is permanent, while the timber lining of a rectangular shaft rarely lasts longer than fifteen years. The Continental practice is to line a circular shaft with bricks. A similar brick lining is largely used in English practice, often in combination with iron. But everywhere the circular lining is of durable materials, that being the only practicable form which lends itself to their use. This alone constitutes an advantage tending strongly to give the preference to the circular shaft. The principal arguments advanced for the rectangular form are that less material has to be removed for a given cage-space and that, in sinking, the permanent lining is at once put in place as the work progresses. But removing the corners in a rectangular sinking is expensive and attended by danger. Moreover, the danger, as well as the difficulty of keeping out the water, is much greater in sinking through quicksand. It is generally conceded that where great pressure is encountered, the circular form is the only safe one. Add that a circular shaft offers less rubbing-surface to the ventilating current, and that the segments at the side of the cages furnish space for the air-current without additional enlargement of the shaft, and there results a preponderant weight of advantages on the side of the circular form of shaft.

G. G. A.

Remarkable Shaft-Sinking Operations.

WILLIAM YOUNG WESTERVELT.

(Engineering and Mining Journal, New York, 1910, vol. lxxxix, pp. 275-6.)

At the Mary mine of the Ducktown Sulphur, Copper and Iron Company near Isabella, Tennessee, a new shaft, required for the extension of already existing workings, is being sunk from several levels simultaneously. The location of the shaft at the surface having been decided on, surveys were made to locate it underground, and drifts and cross-cuts were then driven to that point from five different levels. From each level a "raise" is being excavated on what is believed to be the centre line of the shaft, but only about 6 feet in diameter, sufficient to allow of free delivery of the broken material. This limitation of the diameter allows for several feet of variation, so that no difficulty is anticipated in making the completed shaft come out true to the surface-level. When the "breakthroughs" have been made, these raises will be reamed out to the full size of the shaft. Thus this shaft is being sunk, or worked at, from seven different points in its length, namely, top and bottom and five cross-cuts. The sinking from surface is carried on by two gangs or shifts of eight men each. Two men and a boy, with the usual complement of trammers, constitute the gang per shift underground. Two air-driven hammer-drills are used in a raise, and four 3-inch air-drills in the sinking from the surface. It is estimated that by this method of working the sinking will be completed in about half the time occupied in working only from the surface.

G. G. A.

Strengthening Tubbing in Shafts. Professor HEISE.

(Glückauf, Essen, 1910, pp. 91-3.)

Recent failures of iron tubbing in shaft-sinking have raised the question of the best sectional form of the segments, an urgent one in circumstances where quicksand, or running material of any nature, is met with. Water-pressure can be provided for with some degree of precision. For this it is customary to allow 1,000 kilograms to the square centimetre, and as cast iron has a resistance of 7,500 kilograms, there is a large factor of safety. The danger arises when ground having a tendency to slip is passed through, giving "thrusts" the force of which cannot be estimated. This danger is to be met only by giving the segments plenty of strength. But an increase of the sectional area entails a large increase in cost, and the additional weight makes the segments difficult to handle. Therefore it is desirable to gain strength through improved form. The Author affirms that a rupture in the tubbing is never due to insufficient power of resistance to pressure, but is caused in all cases by a want of tensile strength. Fracture starts on the inner face, and it will take place when the tensile strain there reaches about 1,500 kilograms to the square centimetre. The common form of section may, indeed, be strengthened by widening the flange, but only within very narrow limits. It becomes, therefore, necessary to devise a more suitable form. Three such forms have been proposed, and in some instances, adopted. These are known respectively as the "multiple arch," the "cross-ribbed" and the "corrugated." The capacity for resistance of these several forms of section is discussed, and the conclusion is arrived at that if the breaking-strain of the ordinary sectional form of segment be taken as 100, that for the first of the above-mentioned forms will be 186, that for the second 209, and that for the third 311. Probably the common form would offer a somewhat higher resistance than the first, if it were possible to give it the same depth of flange. Judging these forms from the point of view of facility and certainty in the casting, the corrugated is to be preferred, and for this form the Author appears to decide, though he admits that it does not lend itself so well as the others to the most effective grouping of the bolts.

G. G. A.

Winding-Ropes for Mine-Shafts.

(Oesterreichische Zeitschrift für Berg- und Hüttenwesen, Vienna, 1910, pp. 21–6 and 36–8.)

The Author reviews the experience gained in recent years with wire winding-ropes in mine-shafts, and while indicating the tendencies of present practice, points out practical lessons to be learned therefrom. Iron wire, even of the best charcoal quality, has been tried and found wanting. Its use for shaft winding-ropes has been wholly abandoned, its place being taken by steel, of which the crucible quality is now almost exclusively used. The flat form of rope, too, which a few years ago was very common in Austria and Germany, is rapidly disappearing in consequence of defects which experience has shown to be much more pronounced in the flat than in the round form. Tabulated returns from the Breslau district give precise and important information on this matter. Of the total number of flat ropes reported on, 42·86 per cent. lasted less than 200 days when in constant work, while the remaining 57·14 per cent. had a life of less than 400 days. Comparing these with round ropes working under the same conditions, only 14·29 per cent. and 17·32 per cent. were worn out in less than 200 and 400 days respectively, while 25·54 per cent. lasted 400 to 600 days, 20·34 per cent. 600 to 800 days, 10·82 per cent. 800 to 1,000 days, and the remainder 1,000 to 1,600 days. The inferiority of the flat form is seen yet more plainly when the comparison is in efficiency—that is, work done—expressed in ton-kilometres. For the same quantity of work, the round rope has a 40-per-cent. longer life. The greatest amount of work done in 1908 by a round rope was 376,299 ton-kilometres, while the maximum reached with a flat rope was only 44,544 ton-kilometres. The former, however, was of exceptionally good quality steel, its breaking-strain being 180 to 190 kilograms to the square millimetre, the ordinary standard for crucible steel being 150 kilograms. The tendency of present practice is to increase this, for the stronger rope wears longer, is more efficient and costs less per ton-kilometre of material raised. The latest and most approved quality of rope, now almost exclusively used, is of bright, patent crucible steel, the ordinary crucible-steel galvanized wire being rarely met with. The percentage of broken winding-ropes in mine-shafts in the Breslau district dropped from 9·62 of the number in use in 1882 to 1·26 in 1908, a reduction due chiefly to the substitution of steel for iron and the gradual discarding of the flat form.

G. G. A.

Lubrication of Wire Ropes. R. J. ROWLAND.

(Engineering and Mining Journal, New York, 1910, vol. lxxxix, pp. 379-80.)

The Author, in discussing the wear of wire winding- and hauling-ropes relatively to various arrangements of pulleys, directs attention to the benefits derived from lubricating the ropes used for such purposes, a means of rope-preservation too often overlooked, or only imperfectly applied, especially in the case of haulage operations. The purpose of greasing is not merely to protect the outside of a rope from corrosion, but to lubricate the inner surfaces, and to preserve the hemp-core, which, if allowed to become very dry, will crumble away with disastrous results. The chief use of the lubrication is to lessen the friction of the internal wires which chafe heavily on one another when a loaded rope is passing over a pulley. For this reason it is desirable to work the compound well into the interstices. Apart from the reduction of the sectional area of the wires caused by wear from their constant friction, it is believed that the latter induces fatigue of the steel. An unlubricated rope stood 16,000 bends before breaking, while the same rope when lubricated stood 38,700 bends. Linseed oil is not a good lubricant, because it dries and peels off, and, moreover, the hardening prevents a subsequent application of the oil from reaching the inside of the rope. It has also a tendency to harden the hemp-core and in that way to hasten its destruction.

G. G. A.

Concrete Water-Column.

(Engineering and Mining Journal, New York, 1910, vol. lxxxix, p. 405.)

This application of concrete is described in a notice of a water-column constructed at one of the shafts of the Continental Zinc Company, Joplin, Mo. The water, of which 800 gallons a minute have to be pumped, contains so much sulphuric acid that an iron column lasts only about 30 days. To meet the difficulty recourse was had to concrete. A hole, 16 inches in diameter, was drilled from the surface to a depth of 208 feet, the height to which the water had to be lifted. Into this borehole a 10-inch pipe was inserted, leaving an annular space of 3 inches between the pipe and the solid ground. When this annular space had been filled with neat cement and allowed to set, a solid concrete tube was formed, reaching from the pump-station to the surface. This cement column proved to be a satisfactory solution of the problem.

G. A. G.

Natural Rock Air-Receiver.

(Engineering and Mining Journal, New York, 1910, vol. lxxxix, p. 406.)

At one of the mines in the Rossland district, U.S.A., an unused cross-cut has been turned to useful account as a compressed-air receiver. The cross-cut is roughly 6 feet wide by 9 feet high, and 400 feet in length, and was driven through solid rock showing no cracks or seams. It was proposed to convert this into a compressed-air receiver by sealing it up. This was done by means of a bulk-head 10 feet thick, carefully pointed on the inner face with strong cement. Into this bulk-head, a 6-inch pipe and a manhole were built, the latter being sealed in the same way as a boiler-manhole. Two very small leaks around this manhole account for a loss of only 10 lbs. pressure in 18 hours, the usual degree of compression being 100 lbs. Two electrically-driven compressors, of a combined capacity of 7,500 cubic feet of free air per minute, give this pressure after 23 minutes' working. Allowing one-third of this air to be available at a working-pressure, ten drills can be run 1 hour after the compressors have stopped working. The results are very satisfactory in the steady working of all the machines concerned, including a hoisting-engine and a rope-drive.

G. G. A.

Ball- and Roller-Bearings for Mine-Trucks. SCHULTE.

(Glückauf, Essen, 1910, pp. 240–2.)

The rapidly extending use of ball-bearings is strikingly shown by what appears to be a successful attempt on the part of the Harpener Bergbau-Aktien-Gesellschaft to apply them, in a simplified and strengthened form, to mine-trucks. In spite of the rough usage to which such trucks are subjected, these improved bearings have so far given satisfactory results in practice. Following on the recently introduced roller-bearings for this service, whereby the rolling-friction has been materially lessened, this further step forward indicates the trend of modern engineering practice towards economy of motive power. A series of trials carried out under test conditions with both empty and loaded trucks showed a notable diminution in the resistance to traction relatively to the roller-bearings. These results the Author has tabulated, deducing therefrom the coefficients of friction and comparing the effects in actual work. Relatively to the roller-bearings, these strengthened ball-bearings effect a saving of 23 per cent. in traction-power. The average expenditure of electric current at the Company's pits is 0·142 kilowatt per ton-kilometre with trucks provided with roller-bearings. The cost of current being about ¾*d.* per kilowatt, and the monthly output being represented by 40,000 ton-kilometres, the annual saving will amount to about £43. But from the economical point of view, there is a greater gain from the absence of a need for frequent lubrication. The bearing is so effectually protected from dirt that cleaning and oiling once a year is sufficient. Besides rendering the withdrawal of the trucks from service less frequent, this circumstance leads to a great saving in lubricating materials. The trucks provided with roller-bearings in use at these mines are regularly lubricated once a month. The average quantity of material used being 0·83 kilogram for each truck, the annual cost for the two thousand trucks in service amounts to £204 at present prices. The cost of lubricating for the same time with ball-bearings is only £17. But these advantages in a notable saving in the cost of working and maintenance are lost when account is taken of the greater first cost and its consequences in interest and depreciation. This the Author finds to be the only hindrance to the general adoption of the ball-bearing. To restore the balance in favour of the latter he recommends an increase in the carrying-capacity of the trucks, for with these bearings the resistance does not increase proportionally to the load.

G. G. A.

Electro-Metallurgy at Philadelphia Mint. H. J. SLAKER.

(Electrical World, New York, 1910, vol. lv, pp. 1647-9.)

Within the last ten years the Philadelphia Mint has adopted electrolytic, in place of acid processes, for refining gold and silver bullion. The system now in use is a double one, the first step being to bring the silver up to purity and the second, the gold. In the silver-refining the anode is 40 per cent. gold, 47 per cent. silver and 13 per cent. the base in the original material. Details of the anodes, cell and electrolyte used are given. The gold-refining process is also described, together with the method of recovery of platinum. Details of the equipment are given, and the article is illustrated by photographs.

J. A. S.

Rust on Cast-Iron and Wrought-Iron Pipes.

DR. O. KRÖHNKE.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxxiii, pp. 393-404.)

A brief record is given of the different theories as to rust put forward by Diegel, Heyn, Bauer, Walker and others, the outcome of which is the general recognition of the fact that the production of rust is effected by the agency of electrolysis. This theory has still further gained ground since Walker has proved in contradiction to Newton Friend, the latest advocate of the carbonic-acid-gas theory, that the facts relied on by the latter can be set down to electrolytic action. The present investigations form part of the examination of this question, undertaken since May, 1908, on behalf of an influential body of German pipe-manufacturers, with a view to discover the causes of corrosion and the best methods for the protection of iron pipes. Reference is first made to the electro-osmotic theory of Nernst, in accordance with which all substances possess a definite "solubility-extension" in regard to the solvent fluid. Nernst understands by this expression the expansive force which seeks to introduce the molecules of the substance into the solution. Under the influence of this dissolving pressure the substance, as soon as it has the opportunity, enters into solution and continues to do so until the osmotic pressure of the dissolved portion attains to equilibrium with the solubility-extension of the substance, whereby further solvent action is hindered. It is shown that iron obeys this theory, and the action of the ions is explained. Even the purest water is to some extent in such a condition that it contains a certain proportion of free positively-charged hydrogen ions and negatively-charged hydroxyl ions, and the manner in which iron is dissolved in consequence of ionization is discussed. The electrolytic theory of rusting is supported by what Cushman and Walker have termed the "ferroxyl reaction," which is here fully elucidated, together with the gelatine reactions. Numerous photographs show the effect of special solutions on iron, and the method of the formation of rust, and illustrations are given of the surfaces of different kinds of cast-iron and wrought-iron pipes attacked by rust. Some attempt is made to classify the various characters of rusting, together with the factors which influence the rapidity of the action leading to the speedy decay of pipes, and the results obtained are set forth. The surface-skin or coat of iron castings is shown to possess a marked protective influence.

G. R. R.

Slow-Rusting Iron Produced in the Open-Hearth Furnace.

(Engineering News, New York, 1910, vol. 63, pp. 6–8.)

What has been named "American ingot iron" is a product of the open-hearth steel furnace; but in chemical composition and mechanical properties it is allied more to wrought iron than to steel, in having its carbon, manganese and silicon contents reduced to small fractions of the amounts found in ordinary soft steel. A typical analysis is given. The prime quality of the material is its resistance to corrosion, and this is the main object of its introduction. The manufacture is fully detailed under the headings of raw material, typical melt records, furnaces and ingots, and furnace-wear. Then follow descriptions of the results in uniformity, strength and ductility, which are generally equal to wrought-iron, and resistance to corrosion. The tests in the latter were by means of sulphuric-acid solutions, which are only inferential as regards atmospheric exposure; but as these showed a resistance forty to sixty times as great as steel, there is a large margin for error in the use of inference. Other less important qualities, such as weldability, ease of annealing, etc., are discussed.

C. O. B.

Corrosion of Iron and Steel. Professor HEYN and — BAUER.

(Mitteilungen aus dem k. Materialprüfungsamt, Gross-Lichterfelde, 1910, vol. xxviii, pp. 62–137.)

The Authors have carried out a large number of experiments, of which the first set deals with the relative corrosion of wrought iron and mild steel. Although the uncertainty attaching to the results amounts to 10 per cent. yet on an average the wrought-iron samples corroded at first about 20 per cent. faster than the steel, but after a time the relation was reversed and the iron samples corroded 9 per cent. slower than the steel, and for both metals corrosion was much faster during the first than during the subsequent months.

Corrodibility in acids seems to be no guide as to the corrosion rates in water, in which fluid mild steel, wrought iron and cast iron waste at approximately equal rates. In water saturated with CO_2 the rates of corrosion are as 1·0, 1·31 and 4·30; whereas in a 1-per-cent. solution of H_2SO_4 the rates are as 1, 2 and 100. A few additional salts were tried and it was found that calcium sulphide, if present in large proportions, reduced corrosion. Beet sugar was found to cause pitting, and so did ferri- and ferro-cyanide salts as well as borax, but the worst effects were produced by adding chloride of sodium (common salt) to waters containing soda. In fact, common salt seems to neutralize all the beneficial effects of anti-corrosive salts. The effect of submerging the several metals in running streams of water showed that such differences of electric potential as existed were reduced by velocity, but, on the other hand, under these conditions the protective rust was removed and corrosion proceeded more rapidly than in stationary water. A few preliminary experiments were made with hot solutions of salt and of magnesium chloride, and in both cases corrosion increased with the temperature.

C. E. S.

Transport of Iron Ore from the Canigou to Port Vendres.

J. E. Giraud.

(Le Génie Civil, Paris, 1909, vol. lvi, pp. 81–5.)

The valuable deposits of iron in the Pyrenees have been known for several centuries, the ore in early days having been smelted near the mines, timber growing in the vicinity being used as the fuel. Improvement in metallurgy, together with the growing scarcity of timber, has led to the abandonment of these primitive methods, and owing to the difficulties of transport, especially during the winter, a great number of valuable mines have become disused. With a view to exploiting an important group, the works described were undertaken. The railway-terminus most convenient for the Canigou range is Amélie-les-Bains, whence the distance to the mines, following the tortuous route which had to be adopted, is 11 miles. In this length the total drop is 3,683 feet, and the system of transport adopted is a composite one, a short length (0·87 mile) at the upper end, and about three times this length at the lower end, being of the overhead-ropeway type, while the intervening distance of 7½ miles is traversed by a narrow-gauge railway. As the descending is by far in excess of the ascending load, no power is required to operate either of the ropeways; indeed there is a surplus power of 10 HP. given out in the case of the upper one, and no less than 150 HP. in that of the lower. The maximum span is 1,640 feet, and the speed of travel, which is the same in both ropeways, is 5·59 miles per hour. The 7½ miles of 24·6-inch gauge railway are laid with 30-lb. flat-bottom rails supported on oak sleepers spaced 23 inches apart. Each train carries 60 tons.

The ore on leaving the mine is tipped into large hoppers, from which the upper aerial line is fed, and similar hoppers are provided wherever the system of transport changes, no attempt being made to load wagons direct from skips or vice versâ. Port Vendres forms the natural shipping place, and here storage-hoppers of a capacity of no less than 40,000 tons have been provided. Much of the ore finds its way to England, and, notwithstanding the length of journey, the time which elapses between the mining of the ore in the Pyrenees and the rolling of the finished plate in England is only 16 days.

I. C. B.

Steel Specifications to meet Present Requirements. C. A. Tupper.

(American Machinist, New York, 1910, vol. xxxiii, pp. 534–9.)

This article covers practically the whole field of the use of steel for machinery of every kind, for bridges, rails and construction work, and gives detailed particulars as to elastic limit, breaking-stress, elongation and contraction of area, together with the complete chemical analysis desirable in each case. The effect of small percentages of such metals as nickel, chromium, vanadium, titanium and molybdenum is considered at some length, and several examples are given of some remarkable steels thus produced. One of these may be cited; it is a chrome-vanadium steel suitable for making springs having an elastic limit of 227,100 and a breaking stress of 237,500 lbs. per square inch; an elongation in 2 inches of 10 per cent. and a reduction of area of 35 per cent. The composition of this steel is in percentage: carbon 0·48, manganese 0·90, chromium 1·25 and vanadium 0·18. In connection with rails it is stated that the New York Central Railway has been trying a new steel known as "titanium," the analysis of which gives carbon 0·45, manganese 0·96, silicon 0·13, phosphorus 0·095 per cent. (the percentage of titanium is not given). These rails are characterised by a decided increase in strength over those previously used, and the increase in durability is said to be 300 per cent. A Table gives forty-five different types of steel for a great variety of purposes, stating their mechanical characteristics and chemical composition.

H. R. S.

Titanium-Steel. WILHELM VENATOR.

(Stahl und Eisen, Düsseldorf, 1910, pp. 650-4.)

Titanium as an alloy in steel, until recently regarded by the steelmaker as an undesirable element, is now sought as an efficient means of obtaining metal of superior quality. In the United States titanium-steel rails are rapidly displacing the ordinary kinds. European manufacturers have been slow to follow the United States' lead; perhaps because little is yet known of the way in which titanium acts. The Author adduces evidence of a convincing nature obtained through experiments carried out on practical lines at Osnabrück in connection with the production of Siemens-Martin steel in basic furnaces. To make the tests comparative, a charge was divided into two halves, and to one of these titanium was added in the proportion of 0·038 to 0·14 per cent., and in most of the experiments 0·02 to 0·04 per cent., a mere trace. From these two half-charges, a number of both large and small blocks were cast, and subjected to chemical analysis and mechanical tests. Rolling the blocks into bars proved them to be sound throughout. The results of these tests show conclusively that the addition of titanium to steel affects the qualities of the latter in a marked degree, and that its influence, in proper proportions, is unmistakably beneficial. The effect is to toughen the metal. Very different is the appearance of the fracture, which is fibrous, like that of wrought iron, from that of the unalloyed metal, which is granular. The difference is shown by photographic illustrations. This fracture appears to be characteristic of titanium steel, and is of a "tearing" nature. Bending tests revealed the same increased tenacity. That the addition of these small proportions of titanium causes a far-reaching change in the steel is made strikingly clear. American practice has shown that titanium-steel rails wear more slowly under heavy traffic than those of ordinary steel. The increased tenacity of such steel points to its adoption for all building purposes, but especially for bridge-building. The Author believes that the action of titanium is chiefly one of deoxidation. It is not an alloying, but apparently a purifying substance. Whether or not it serves to expel nitrogen can be determined only by further research.

G. G. A.

Resistivity of Nickel, Commercial Aluminium and their Alloys.

H. PECHEUX.

(La Lumière Electrique, Paris, 1910, vol. x, pp. 232-9.)

After discussing the method of obtaining nickel and aluminium, the Author gives the results of recent researches on the resistivity of these metals and their alloys with copper, with a view of indicating their uses in the construction of resistances for industrial purposes. The resistances of spirals heated in a paraffin-bath were determined by the Wheatstone-bridge method. In the case of nickel-copper alloys the temperature-limits within which the variations of resistivity follow a parabolic law are determined and curves are given. Cupro-nickels, zinc-nickel-copper alloys, and iron-nickel are also dealt with. Alloys are electrically preferable to metals as resistances when they have a high resistance and small volume and when they have a temperature-coefficient as small as possible. Cupro-nickel and the zinc-nickel-copper alloys are good in this connection. Three mixtures of aluminium were also studied between 0° and 370°, and temperature-resistance curves are given. Deductions as regards the suitability of aluminium and cupro-aluminium for electrical work are made.

J. A. S.

Dutch Commission on White Lead.[1] *Final Report.*

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 73-77.)

This commission was formed to investigate to what extent white-lead paints can be replaced by white zinc or some other substance. The causes of the deterioration of white lead are fully discussed. The main objection to the use of white zinc appears to be that it does not adhere as intimately to wood or ironwork as white lead. It is also alleged that white zinc does not cover as well as white lead. Tests were made on fresh and on previously-painted wood, on plaster, zinc, iron structures, wood exposed to damp sea-air, iron exposed to sea-air, and lastly, on internal woodwork. The paint was tested by first cleaning it carefully with a soft sponge and water, and subsequently giving it twenty strokes with a fairly hard flat brush thoroughly moistened. The brush was then shaken over a trough with clear water. If any paint had come away with the brush it would be shown by the formation of clouds in the water. The final conclusions of the commission may be summed up as follows:—With regard to covering power, white zinc compares favourably with white lead. White-zinc paint resists sulphuretted hydrogen better than lead paint, but it does not resist vapours containing sulphuric acid nearly so well. Zinc paints are equal to lead paints in resisting ordinary atmospheric conditions when applied to iron, cement or zinc. This also applies to all indoor work, and in these special cases zinc paints could readily replace lead paints. White zinc does not last nearly so well as white lead. It is more costly to cover old paint work with white zinc than with white lead. Lithopone was found to be quite unsuited for outdoor work. For any work above water iron paints can be readily substituted for lead paints to serve as a priming; the cost will thus be decreased, but the former paint requires greater skill in application and also takes much longer to dry hard. The commission appears to favour the use of zinc paints in many instances where lead paints are being used at present.

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[1] See *De Ingenieur*, The Hague, 1907, p. 137, and 1908, p. 219.

H. C. A. T.

Monel Metal. A. Stansfield.

(Transactions of the Canadian Society of Civil Engineers, Montreal, 1909, vol. xxiii, pp. 302-19.)

The two metals nickel and copper are associated in large pyrrhotite deposits, and it has been found difficult to separate them. For many purposes it is found to be unnecessary to separate them, and an alloy may be obtained combining them in the same proportion as they occur in the ore. This alloy, called "Monel Metal," can be used as a basis for any useful alloy such as for coinage (75 per cent. copper and 25 per cent. nickel), and is found to possess such valuable properties that it can be widely used. The Author describes the method of manufacture and the properties, which vary with the amount of carbon, iron and sulphur, etc., and with the pouring temperature. The results of tension-tests on hot-rolled bars show that the metal has a yield-point of 76,300 lbs. per square inch, an elongation of 30·75 per cent. and a reduction of area of 49·3 per cent. An annealed specimen gave a yield-point of 38,200 lbs. per square inch. For the cast metal the yield-point was 21,900 to 31,500 lbs. per square inch. These results are to be regarded as individual tests rather than averages. R. S. B.

Progress in Electrostatic Separation. H. A. Wentworth.

(Engineering and Mining Journal, New York, 1910, vol. xc, pp. 15–17.)

In March, 1908, an electrostatic plant for zinc-separation on a commercial scale was set up for the American Zinc, Lead and Smelting Company at Platteville, Wis., where it is now working successfully. Since that date further progress in design and construction has brought this class of machine into rapidly extending use. It is a common error to confuse electrostatic with magnetic separation, which is based on wholly different principles. To effect the separation of minerals in ores the former utilizes their difference of conductivity. When a mixture of minerals is subjected to electrical influence there are differences of time required to charge each of them, and also, when charged, there will be differences in the time of discharge when the minerals come into contact with a discharging surface. These are the fundamental principles on which all electrostatic separators act. The latest developments in this class of machinery are the removal of the necessity for close supervision, the widening of the range of size of materials dealt with, and a marked advance in the separation of minerals of nearly the same conductivity. An electrostatic separator now requires no more attention than a concentrating table, and the range of size is from what passes through a 200-mesh screen up to pieces ⅓ inch diameter. The capacity of a separator of this nature varies with the size of the particles, the difference in conductivity of the minerals and the grade of the product desired. With the Huff machine it ranges between 4 or 5 tons when working on the finer materials and 15 or 16 tons on coarser sizes. The mechanical and electrical power required averages about ⅓ HP. for each machine. The Huff separators, of which an illustration is given, are constructed almost wholly of metal and without shaking parts. The Author states that this electrostatic separator may be used as a general concentrator of sulphide ores from their gangues, especially where water is scarce, as a separator of copper, zinc and lead-silver ores from heavy gangues, as a concentrator of molybdenite, graphite or monozite, for the solution of all other special problems in which water-separation alone is not successful, and as a concentrator of pyrites for sulphuric acid.

G. G. A.

Differential Pressure-Gauge in Engineering Thermometry.

J. A. Smith.

(Proceedings Victorian Institute of Engineers, Melbourne, 1910, pp. 3–8.)

It is obvious that the difference of temperature between two points can be measured if at each point there is the bulb of a gas-thermometer and these bulbs are suitably connected by pipes to a differential pressure-gauge of the barometric type. The practical difficulty is the insensitiveness of this form of pressure-gauge, but this difficulty has been eliminated by applying the sensitive two-fluid gauge previously described by the same Author.[1] By this means it is possible to detect temperature differences of $\frac{1}{500}$° F. and measure differences of $\frac{1}{200}$° F. The two-fluid differential gauge is formed of a glass tube 10 millimetres bore and 37 inches long, connected at its bottom end to a glass tube 3·3 millimetres bore and forming a U-tube with it. The smaller tube is connected at its upper extremity to a piece of 10-millimetre tube 5 inches long. Alcohol and water slightly coloured and of a specific gravity of 0·900 is poured into the larger tube, and kerosene oil of 0·825 specific gravity is poured into the other limb of the U-tube. The two liquids will not mix and their junction can be adjusted to any desired point of the smaller tube and forms the zero when the two extremities of the U-tube are at the same pressure. Two metal sliding gauges are attached to this tube; one marks the zero and the other the movement of the junction of the liquids due to a difference of pressure, and this movement can be measured by means of an equally-divided scale. A thick layer of oil is placed on the top of the alcohol to prevent evaporation, since the vapour-tension would produce an error in the corresponding gas-thermometer. The bulbs of the gas-thermometers can be made of iron or of invar, and of any required shape according to the purpose desired. The connections are made by brass tubes of $\frac{1}{50}$-inch internal diameter, and, in order to correct errors caused by these tubes, they can be balanced by duplicate tubes applied in the same manner as the balancing leads of a Callendar platinum thermometer. The calibration is effected as follows:—"Define the position of the common surface of the liquids when both limbs are freely open to the atmosphere. Add to the spirit side spirit equal to a definite length of that tube. Then the resultant movement of the common surface, divided by that added length reduced to the equivalent length of an equal weight of water," equals the amplification in comparison with a simple water column. In the actual instrument exhibited at the meeting 1° F. was represented by a movement of 4½ inches of the common surface.

H. R. S.

Nitrogen Thermometer from Zinc to Palladium.

A. L. Day and R. B. Sosman.

(American Journal of Science, New Haven, 1910, vol. xxix, pp. 93-261.)

Over 5 years ago the Geophysical Laboratory undertook to redetermine the absolute temperature-scale from 300° to 1,100° C. with a nitrogen thermometer, and the work done in this connection was published in 1908.[1] In this article is described the continuation of the work, extending the nitrogen scale up to nearly 1,600° C. The apparatus used is described in some detail, and the errors involved in the measurements and the methods of evaluating these errors are gone into very fully. For instance, the error due to the want of uniformity of the temperature of the bulb was determined by means of a large number of thermo-elements, consisting of platinum wires attached to different points of the bulb, and by this means differences of temperature of as much as 8° were observed; the estimated error of the mean temperature of the bulb is, however, only 0·2° at 400° C. and 0·4° at 1,500° C. A complete list of the errors and their estimated amount is given. Since the bulb of the nitrogen thermometer cannot be placed into melting or solidifying substances at high temperatures, the determination of these melting-points, to be used as "fixed points," had to be effected by transference by means of a thermo-element. The article contains lengthy Tables giving full particulars of the results of the observations, and there is also a complete account of the chemical analysis carried out to determine the impurities in the various "pure" metals used as fixed points. The following are some of the melting-points (degrees Centigrade) obtained, the figures in brackets being the Reichs-Anstalt scale: zinc, 418·2 ± 0·3 (419·0); silver, 960·0 ± 0·7 (961·5); copper 1,082·6 ± 0·8 (1,084·1); platinum 1,755; aluminium 658·0 ± 0·6 (657·0).

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[1] Minutes of Proceedings Inst. C.E., vol. clxxx, p. 406.

H. R. S.

Platinum Resistance Thermometry at High Temperatures.

C. W. WAIDNER and G. K. BURGESS.

(Bulletin of the Bureau of Standards, Washington. 1909, vol. vi, pp. 149-230.)

The facilities at the disposal of the Authors have enabled them to study this question with great exactness and they have been able to determine the precision with which high-temperature measurements can be made, together with the effect of impurities in the platinum wire and the changes due to high temperatures. Nine platinum thermometers were used, made with wire of varying degrees of purity and of various diameters and resistances; two palladium thermometers were also used. The method of construction of these thermometers is fully described, as well as that of the electric furnace employed for melting the various metals, the freezing-points of which were determined, namely, tin cadmium, lead, zinc, antimony, silver and copper. Samples of pure metal were obtained from different sources, and in most cases concordant results were obtained; with antimony, however, minute amounts of impurity greatly affected the freezing-point. When the calibration data for each thermometer were arranged chronologically the changes in the thermometric constants, due to the temperatures to which the thermometer had been exposed, became apparent, and showed very clearly the advantage of using wire of pure platinum. In determining the freezing-points at the higher temperatures, it was found necessary to calibrate the thermometer both before and after the measurement, and unless this precaution be taken the precision of the observations is much reduced. The boiling-point of sulphur was also determined for calibration purposes, and after an extensive series of experiments with an iron-tube apparatus of the Callendar and Griffiths type a glass-tube apparatus was adopted. After reviewing the work of other experimenters, the Authors assumed the boiling-point of sulphur to be 444·7° C., and this gives 419·37° C. as the freezing-point of zinc according to their experiments. Day and Clement, however, have recently obtained with a nitrogen thermometer 418·5° C. as the freezing-point of zinc, showing a disparity of nearly one degree. The Authors do not, however, consider that the evidence is sufficient to warrant a change in the accepted value of the boiling-point of sulphur. A bibliography is given of works since 1858 on electrical resistance thermometry and the determination of the boiling-point of sulphur.

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[1] American Journal of Science, 1908, vol. xxvi, p. 405.

H. R. S.

Efficiency of Welded Joints.[1] CH. FRÉMONT.

(Le Génie Civil, Paris, 1910, vol. lvi, pp. 318–25.)

A welded joint can never possess per unit area the strength of the pieces which it couples together because the inevitable presence of scale makes it impossible for the weld to be closed at all points; since most joints, however, are subjected to variation of load, strength is a secondary consideration as compared with toughness. The Author has demonstrated in a series of tests made in the impact-machine that welded joints are singularly deficient in toughness. The bars which were employed for the test samples measured 0·394 by 0·315 inch in section and the welds were made both on the anvil and by the autogenous process. While 361·65 foot-lbs. of work were needed to bend double the unwelded portion of the bar and this without breaking it, 7·23 to 115·73 foot-lbs. only were required to break apart a weld made on the anvil, and less than 15 foot-lbs. in the case of the best weld made by the autogenous process. These figures indicate that (1) the toughness of the weld, even when made by the normal process, is exceedingly low as compared with that of the bars united by it; (2) the autogenous weld from the point of view of toughness is greatly inferior to that made by the normal process; (3) great variation in the strength of welds is found even when they are made by identical methods. External appearance is no guide to the strength of a welded joint. In the impact-tests made on autogenous welds it was usually found that the break occurred along the plane of the weld itself and not through any of the solid material. It has been claimed that autogenous welding is especially suitable for boiler-repairs as the brittleness disappears when the joint is heated to the temperature of steam at pressures commonly in use, but the Author's experiments, although made on samples welded under the best conditions by the most experienced workmen, do not bear this out. Reproductions of photographs include some depicting etched sections of welds, which show want of homogeneity, especially in such welds as have been effected by the autogenous process. It is pointed out in conclusion that, although certain boiler-repairs of a minor character may be efficiently carried out by autogenous methods, work of such extent that a failure would result in disaster should not be done in this way.

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[1] In a subsequent issue of this journal (pp. 392–3) appears a letter, signed "A. Le Chatelier," protesting against the somewhat alarmist view expressed in the article from which this abstract is taken, followed by a reply from the Author.—I. C. B.

I. C. B.

Manufacture of Submarine and Submersible Boats. LAUBEUF.

(Mémoires et Travaux de la Société des Ingenieurs Civils de France, Paris, 1910, vol. i, pp. 81-90.)

In 1902 France was in the exceptional position of possessing fourteen submarines and submersibles, with twenty-four submarines and thirteen submersibles in construction. England had five and one in construction, the United States six and two in construction, Italy had one and the other Powers none. Now England has seventy-seven and France seventy-nine, England having sixty afloat and France fifty-five. Russia has thirty-one, but has not laid any down for 3 years. The United States have twenty-eight, Austria has seven, Italy has seven and twelve more projected, and Germany has voted annually since 1907 increasing sums for submarines. One cause of the want of progress in France is that the building of submarines is confined exclusively to the arsenals and the work is not pushed on. The first submersible boat, the *Narval* was built by France in 1898. The main differences between submarines and submersibles are in the position of the water-ballast tanks, which are inside the hull in submarines and outside in submersibles; in the coefficient of floatability, or ratio of emerging to total volume, which is 5, 7, or 10 per cent. in submarines and about 30 per cent. in the Laubeuf type of submersible, and even 41 per cent. in the *Narval:* and in the shape, submarines being circular in section and pointed at the ends, while submersibles are shaped like torpedo-boats. The great reproach against submersibles at the outset was that in passing from the surface to a state of immersion they were in a hybrid position and liable to destruction by an enemy without the possibility of escape. The 28 minutes required for the first plunges of the *Narval* have been reduced to 12 minutes, and 5 minutes is the time needed by the latest type, the *Pluviose*, 1905. A list of types of French, English and American submarines and submersibles is given. In France there are forty-seven submersibles completed or in course of construction, while Germany, Italy, Sweden, Norway and Denmark have adopted the type exclusively. Austria and Russia build submarines and submersibles.

W. H. B.

Electric Equipment of the New French Submersibles.

J. Breguet.

(La Lumière Électrique, Paris, 1910, vol. 9, pp. 35–40 and 67–73.)

The Author deals with such details as can be made public regarding the electrical equipment of the *Mariotte*, the *Archimède* and the *Amiral Bourgois*, the largest submarines yet constructed. The boats have a displacement of about 800 tons, are driven by engines of 2,000 HP. (French) adapted for operating, on the surface and submerged, at a speed of 10 knots. Each boat has two propellers, each shaft having an electric motor in addition to a steam or explosive engine. The relative applicability of speed-control by rheostat-control in the armature-circuit and of varying the excitation is exhaustively discussed, leading to a demonstration of series parallel control. The solution of the problem adopted on the *Mariotte* by coupling batteries in series and parallel, and on the *Archimède* by coupling armatures in series and parallel, is described, and the use of the motors as reversible machines for recharging the batteries is developed.

J. A. S.

Salvage Work and Wreck-Raising. Ribière.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xliv, pp. 185–91.)

In wreck-raising, the vessel is either lifted by being slung from lighters, or, after having been made staunch, is pumped out, when she floats by her own buoyancy. In the former case chains or wire-ropes form the slings which are either passed under the wreck or attached to suitable points on it. In the latter case when operations are being conducted in comparatively deep water, i.e., when the wreck is submerged even at low tide, an encircling dam attached at its lower end to the hull, is required, and the construction of this necessitates extremely careful work on the part of the divers Two appliances have been introduced which have greatly added to the range of the operations capable of being efficiently performed

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by divers:—(1) The electric lamp which can burn under water and thus enables every crevice to be examined minutely, and (2) pneumatic tools which make it possible to cut or drill the iron plates quickly and without excessive fatigue to the diver. When once the leaks have been made staunch the success of the operation depends on the sufficiency of the pumping-plant. Extensive operations in wreck-raising are usually entrusted to firms who have at their command vessels fitted with numerous and costly appliances, and it is not suggested that dock authorities should supply themselves with such salvage-vessels, but it is pointed out that a great variety of wreck-raising operations can be carried out with the general plant found at all ports, while the floating workshop of the salvage-vessel is unnecessary except at a distance from home, i.e., for work which would not be undertaken by a dock authority. As regards special appliances, a few lighters fitted with lifting-gear might be added to the plant, and the ordinary fire-float, if provided with a quick-steaming boiler and a centrifugal pump capable of discharging, say, 220,000 gallons per hour, will furnish a highly efficient tool. In addition to plant, experienced divers and workmen must be available.

I. C. B.

Relation between River and Subsoil Water-Level.

Dr. Ing. G. Thiem.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxxiii, pp. 377–83.)

The Author discusses the fluctuations observed in a tract of country, known as the Fiener Bruch, of the water-line in brooks and the levels of subsoil-water. The character of the locality is explained by a sketch map, and by an imaginary section which shows the nature of the stratification. The investigations here recorded were undertaken in connection with the search for a water-supply for the town of Magdeburg, which has been long under consideration. It is wished to substitute for the existing supply, drawn from surface-water, an underground source of derivation. The Fiener Bruch is stated to be an ancient dried bed of one arm of the Oder, and it was deemed expedient to ascertain whether or not the general direction of the flow of the subsoil-water, which has here a trend towards the valley of the Elbe, into which the Oder formerly flowed, still maintains the original course of the latter river, that is, with a flow from east to west. The whole area under notice comprises about 116 square miles, and by means of numerous observations the direction, which is invariably northwards, of the considerable volume of water in the subsoil was plotted. The Author explains also the course of certain brooks traversing the district which have a sufficient volume of water to turn mills, and indicates by graphic diagrams the rain or snowfall, the water-levels in the rivulets and in the subsoil for the months of December to April, 1908–9. It is evident from the diagrams that the water-level in the brooks largely depends on the rainfall, and the same is true of the general water-line in the subsoil; but a lower level is also shown which indicates the water in the deeper layers of the soil not influenced by the rainfall. By means of bore-holes it was ascertained that a point is soon reached near the watercourses where the flow of water in the subsoil is entirely distinct from that in the brooks. In many cases it was possible by means of temperature-observations to detect the inflow of the surface-water, and the chemical analyses of the two kinds of water prove the qualities of each to be quite distinct.

G. R. R.

Artesian Well at Maisons-Laffitte.

E.-A. A. Martel and Paul Lemoine.

(La Nature, Paris, 1910, Premier Semestre, pp. 194-5.)

This well, 1,889·8 feet in depth, completed in April 1909, at Maisons-Laffitte, was begun in August 1907, at the height of 85·3 feet above the left bank of the Seine, penetrated the various beds of the Eocene formation and reached the chalk at a depth of 209·5 feet. The water-bearing strata of the greensand were attained at a depth of 1,870 feet, and here the flow of water amounted at first to 4,400 to 6,600 gallons per hour. On reaching a further depth of 9·8 feet, the hourly yield of water rose to 37,400 gallons. On attaining the depth of 1,884·5 feet, a bed of hard sandstone conglomerate, containing pyrites and 40 to 60 inches in thickness, was encountered, and on piercing this stratum, as was also the case in the well at Passy, the maximum flow was met with. A powerful outrush of water burst forth, arched at the summit in the form of a paraboloid, 28 inches in height. The daily yield, which, in the first instance, was estimated at 3,520,000 gallons, has since been maintained at 3,080,000 gallons. The temperature of the water is 79·7° F. It is pointed out that there are two supplies, the one above and the other below the bed of sandstone, the latter being much the more copious. Their mineral contents are approximately identical, and it is not known whether the two springs are separate in all places, or whether the stone floor may exist only in patches. The water is pure and limpid and slightly alkaline, but the temperature is such as to render it unsuitable for drinking or domestic uses. None of the artesian wells in the vicinity of Paris yield water cool enough for potable supplies. An account is given of all the different wells hitherto sunk into the greensand below Paris, with a warning, based on experience gained in the United States, of the danger of multiplying the number of these wells, in view of the probable limitation of the yield and the ultimate exhaustion of the springs. All these artesian wells in the Seine basin draw their water from the greensand, and it is affirmed that enquiry should be instituted into the general policy of further sinkings. The whole question is said to be one worthy of serious attention. G. R. R.

Chaudière Dam, Ottawa, Canada. D. L. McLean.

(Engineering News, New York, 1910, vol. 63, pp. 741-3.)

This regulating stop-log dam, built just above the crest of the Chaudière Falls on the Ottawa River, is of steel and concrete construction, and of the pier and sluice type, the sluice-openings being provided with stop-logs moved by a powerful electric winch travelling on rails on the bridge-floor of the dam. It is formed in plan of a segment of a circle of 566 feet 4 inches radius, with forty-nine piers and two abutments, the length being 1,300 feet. An Engineering Commission, after inspecting several works of this kind in the United States, recommended this form. The drainage-area, rainfall, discharge at various seasons, and backwater effect, are dealt with, followed by a description of the piers which are tapering in plan, and the reasons for their special design, in connection with the water- and ice-pressure to be withstood, and the clearance for debris. A plan and sectional elevation show the reinforcing arrangements. The bridge-floor, 23 feet 5½ inches by 15 inches, is also of reinforced concrete. The stop-log winch has been found effective in the severest weather.

C. O. B.

Reconnaissance Method of Determining the Volume of Reservoir-sites. H. W. Sheley.

(Engineering Record, New York, 1910, vol. 61, pp. 581-4.)

The estimation of the surface-water area being easily obtained by well-known methods, the Author deals with the mean depth, for which he uses a special system illustrated by a comprehensive list of thirty-nine actual surveyed sites in America, giving the varying coefficient of the maximum depth which he proposes to apply to reservoirs of different character. Maps with contours of three of these are given. Firstly, the pyramidal shape is dealt with, which the Author regards as that produced in the earliest geological formation of a basin in combination with the dam erected to conserve the drainage. Here the approximate volume is evidently the area of the water-surface multiplied by one-third of the maximum depth, so that the coefficient is 0·33. About seventeen of the examples in the list, including the great Shoshone reservoir of 393,126 acre-feet, approximate to this. Next, there is the case where there are parabolic-shaped cross-sections, of which the coefficient or constant is 0·44. About ten of the examples of this character, including a 2,199,350-acre-foot reservoir, justify the employment of constants of 0·39 to 0·50. Then finally, there is the wedge-shaped, which gives a constant of 0·67, but as the floor is seldom quite level, or the sides perpendicular, the Author estimates 0·55 as best for this class. The remaining twelve are of this character, very few rising higher than 0·60.

In order to prove further the harmony between estimate and result in this matter, several lower contours of the same thirty-nine reservoirs are given with coefficients, the employment of which would have led to the capacity-results actually obtained up to the corresponding levels, these coefficients approximating to the characteristic estimation advocated in each case.

C. O. B.

Pumping-Machinery for Copenhagen Waterworks. A. OLIVARIUS.

(Ingeniøren, Copenhagen, 1910, vol. 19, pp. 75–82.)

This is believed to be the first installation on a fairly large scale of Diesel oil-motors driving centrifugal pumps by means of belts. There are six motors, two of 150 HP. each for the high-pressure pumps, two of 45 HP. each for the well-pumps, and also two of 18 HP. each coupled direct to dynamos. The oil-fuel is stored in two tanks, each of 150 tons capacity; great care has been taken to give accessibility for inspection and repairs of leakage of the tanks and oil connecting-pipes. The oil gravitates from the storage-tanks into a measuring-tank, from which it is forced by compressed-air into the tanks in the engine-room, where it passes through a filter to the fuel-pumps on the engines. The oil thickens at about 18° F. and steam-heating apparatus is provided to keep it fluid in winter. The special design of the centrifugal pumps is described in detail. The exhaust from the motors can be used for heating purposes during cold weather. The cooling-water is taken from the pressure-main and after leaving the motors at about 104° F. is returned to the pump-wells. The electric installation provides light as well as power for the auxiliary machinery, which is all driven electrically. A tabular statement of the tests of the machinery is given. The installation has been at work for about a year, periodically night and day, and has been satisfactory in every respect. The Paper is accompanied by photographs and drawings.

L. S. Z.

High-Duty Waterworks Pumping-Engines.

(Engineering News, New York, 1910, vol. 63, pp. 120–1.)

Descriptions of pumping-engines at Albany, New York, and at Milwaukee, Wisconsin, with duty-test of the former and duty-test and station-duty of the latter are given. Each of the two Albany engines was guaranteed to develop a duty (on test based on plunger-displacement) of 175,000,000 foot-lbs. per 1,000 lbs. of steam 98½ per cent. dry. The high-pressure, intermediate and low-pressure cylinders are respectively 28, 54, and 80 inches in diameter; plungers 25 inches in diameter; stroke 60 inches. The fly-wheels are 18 feet in diameter, mounted on a 19-inch crank-shaft. The suction- and discharge-pipes are 30 inches in diameter. A Table gives figures for each engine with details of pressure, temperature, revolutions, weight of feed-water, water pumped in 24 hours, and other particulars corresponding to a duty in the two engines of 183,115,000 and 178,106,000 foot-lbs. per 1,000 lbs. of dry steam respectively.

The two Milwaukee engines are vertical triple-expansion condensing-engines of the direct-acting crank and fly-wheel, three-plunger type. They were designed to pump 20,000,000 gallons per 24 hours when running at 20 revolutions per minute against a pressure of 67 lbs. and with a steam-pressure not exceeding 125 lbs. at the throttle. The cylinders are 30, 52 and 80 inches in diameter, plungers 33¾ inches in diameter, stroke 60 inches. A Table is appended giving a number of data in the duty-test of each, the duty per 1,000 lbs. of dry steam showing 164,616,967 and 170,691,044 foot-lbs. respectively. The article is fully illustrated.

C. O. B.

Ozone Waterworks of Hermannstadt. Dr. G. ERLWEIN.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxxiii, pp. 457–61.)

The town of Hermannstadt in Hungary has suffered from a series of severe outbreaks of typhoid. The epidemic being traced to the water-supply, the municipal authorities resolved to construct works in which the whole of the drinking-water might be completely sterilized by ozone-treatment, in a similar way to the water-supply of Paderborn,[1] where, by the introduction of the ozone process, typhoid, formerly endemic, has been banished. Based on the results of the recent experiments at Paris, it was decided to adopt the combined system of Siemens and de Frise, by which means it became possible to avoid certain complications, and, without any costly structural alterations, to interpose the tower-apparatus before the water was raised to the high-level reservoir. The arrangement of the plant resembles the plan which has been accepted at the Paris works at St. Maur for dealing with 440,000 gallons per hour. Its characteristic features are that the ozone battery is composed of Siemens-type tubes, while the sterilization-towers are on the Tindal-de Frise plan. In the Paderborn works these towers are filled with broken stone, and act more in the nature of scrubbers, while in the de Frise towers the water streams though the space in full volume, and is subjected to a current of air charged with ozone-gas. No rapid filter is here used, as the water is cleansed by natural filtration at a distance of several miles from the works. The plant is so designed that it has double the capacity needed. The expenditure was £5,250 and by the use of 25 HP. it has been possible to carry on the treatment day and night without intermission. The water has been found in all cases germ-free.

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[1] Minutes of Proceedings Inst. C.E., vol. clxvi, p. 452.

G. R. R.

Coagulating-Plant of the St. Louis Waterworks.

A. J. JACOBS.

(Journal of the Association of Engineering Societies, Boston, Mass., 1910, vol. xliv, pp. 181-205.)

This plant at the low-service pumping-station at the Chain of Rocks, where the entire supply for the city of St. Louis is taken from the Mississippi River, consists of (1) the coagulant-house where the chemicals are stored and prepared for application to the water, and (2) the settling-basins where the coagulum and sediment are deposited. The coagulant-house and the process which it serves are unique in many respects, for there are no filters used, the process being strictly one of coagulation and sedimentation; the quantity of material used is very large, and the entire process, from the unloading of the material to its application to the water, is entirely automatic. The process consists in the addition to the water of a solution of sulphate of iron and a mixture of slaked lime and water called milk of lime. The average amount in grains per gallon of the former is 2·41 and of the latter 5·58, the figures referring to the consumption in 1908. The average daily quantity of lime is 35·1 tons and of iron 12·1 tons. The new building is designed to handle coagulant for an average daily pumping of 133,000,000 gallons and a maximum of 200,000,000 gallons. It is divided into three sections, the storage-bins comprising the two end sections, with the operating section between them. The bins have a capacity of approximately 10,800 cubic feet, giving a total storage of 1,250 tons of lime and 1,600 tons of iron. In the basement the crushers for both lime and iron, the conveyers, the air-compressor and pump are located. On the mixing-floor are the tanks for slaking lime and making the sulphate-of-iron solution, with the auxiliary storage-tanks and weighing and measuring devices. There are three lime-slaking tanks 7 feet 6 inches in diameter by 3 feet 6 inches deep. Above these tanks, and between the four columns supporting the daily-supply hopper, there is a platform on which three automatic scales are set, which weigh out and discharge into the tank the proper amount of lime in a given time. Two men can operate the plant, though it is advisable to have a larger force, especially at night. The motive power is derived from electric motors working on a 500-volt circuit.

R. S. B.

Concrete Water-Tower, Victoria, British Columbia.

A. Kempkey.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 176–87.)

This concrete water-tower, constructed to supply the town of Victoria, British Columbia, with a high-pressure water-supply, was built as part of an improved system of waterworks by which the supply is taken from Elk Lake, about 5 miles north of the city. The tank on the top of the tower, of 83,000 gallons capacity, is supplied by two electrically-driven triplex pumps, each having a capacity of 83,000 gallons per 24 hours against a dynamic head of 150 feet, and arranged to start and stop automatically with a variation of 3 feet in water-level in the tank. These pumps are situated about 1 mile from the tower and are controlled by a float-operated auto-starter in the base of the tower. The tower is on the top of the highest hill in the town in a residential district, which necessitated a structure possessing a pleasing appearance. It consists of a hollow cylinder of plain concrete, 109 feet high, with an inside diameter of 22 feet. The walls are 10 inches thick for the first 70 feet and 6 inches thick for the remaining 39 feet. A steel tank is embedded in the upper 40 feet of this cylinder, and to form the bottom of the tank a plain concrete dome is thrown across the cylinder at a point 70 feet from the base, the thrust of the dome being taken up by two steel rings 14 inches by $\frac{1}{2}$ inch and 18 inches by $\frac{3}{8}$ inch, bedded into the walls of the tower. The tank is covered with a roof of reinforced concrete 4 inches thick. The concrete used was 1 : 3 : 6 for the sub-base and taper base, 1 : 3 : 5 for the tower, and 1 : 2 : 4 for the dome and roof. The cost of the concrete per cubic yard was £5 3*s.* 7*d.*, which the Author states is 25 per cent. more than that of work executed under more favourable conditions. The total cost of the tower was £3,420.

R. S. B.

El Paso and South Western Railway Water-Supply.

J. L. Campbell.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 531–52.)

The El Paso and South Western Railway traverses the arid country of New Mexico, Texas and Arizona, and the water-supply serving that division of the railway lying between Carrizozo and Santa Rosa, a distance of 128 miles, is here described. The average annual rainfall is 9·84 inches, while in 1909 it was less than 5 inches. East and west of El Paso, for distances of 270 miles in each direction, the railway does not cross any streams and the supply of water was obtained from wells 100 to 1,100 feet in depth. On the division served by the new supply this well-water was of very bad quality, the incrusting solids in grains per gallon being in some cases as much as 170. The Author was directed to establish an efficient system of water-supply, taken from the South Fork of Bonito Creek, which flows down the eastern slope of White Mountain (12,000 feet high) and 16 miles south of Carrizozo. This yields an excellent water with only 7·3 grains of incrusting impurities per gallon. The water is conveyed along the railway in 116 miles of wooden pipe and 19 miles of iron pipe, the works including one 350,000,000-gallon storage-reservoir, four 2,150,000-gallon service-reservoirs, and two pumping-plants in duplicate. From a small concrete dam across the stream at an elevation of 7,728 feet, the pipe-line runs down the valley eastward for 5·5 miles to an elevation of 6,980 feet and then 6 miles farther to the storage-reservoir, from which it drops to the Carrizozo Plain. The pressure in the pipe is limited to 130 lbs. per square inch except on 19 miles of the pump-main between Coyote and Corona. Machine-made, spirally-wound, wood-stave pipe, made in sections 8 to 12 feet long, is used with the exterior surface asphalted. The pipe is wound with special flat steel bands, and the total cost of laying 384,300 feet of wooden pipe 11 to 3½ inches in diameter was 2·36*d.* per foot, while the cost of laying 101,300 feet of 12-inch cast-iron pipe was 11·75*d.* per foot. Tests were made of the velocity of flow through the system by introducing bran into the pipe and noting the time it took to traverse the length.

R. S. B.

Cut-and-Cover Construction on the Catskill Aqueduct.

(Engineering Record, New York, 1910, vol. 61, pp. 36–9.)

This work, forming part of the New York water-supply scheme, is of concrete, 17 feet high and 17 feet 6 inches wide, with a capacity of 500,000,000 gallons daily when flowing nine-tenths full. The most economical conditions were found to be when, in earth, the invert grade was about 12 feet below the original ground surface. In locating the line, therefore, the economical contour attained by this condition was, as much as possible, adhered to. A description of the varying classes of cut-and-cover is given with the reasons governing them, the method of building the concrete, its composition and the collapsible steel centering used for the purpose, and full details of the plant. The article is fully illustrated.

C. O. B.

Wallkill Pressure-Tunnel of the Catskill Aqueduct.

(Engineering Record, New York, 1910, vol. 61, pp. 450-4.)

The 92-mile aqueduct for the additional water-supply of New York from the Catskill Mountains, has three chief forms, cut-and cover,[1] ordinary tunnelling, which two correspond with the hydraulic gradient, and pressure-tunnelling where deep valleys or rivers have to be crossed under below the hydraulic gradient and upward pressure has consequently to be provided against. When the depression is not considerable, steel pipes are used, but otherwise, as in the Wallkill River valley, where the tunnel described is 23,391 feet long and 14 feet 6 inches inside diameter, concrete is used. The ability of the material overlying the tunnel to resist the upward pressure was thoroughly tested by borings, the results of which are given. Six shafts, 350 to 480 feet deep, are being sunk, of which three are for construction purposes only, and will be filled afterwards and sealed with concrete: the three others, which are concrete-lined, being the down-take and up-take shafts at each end and one permanent shaft near the river for emptying the siphon should it be necessary. Power-plant furnishes means for carrying on the shaft-sinking, tunnelling, hoisting, lighting, pumping, etc. The article is fully illustrated and the number of men employed, rate of progress and quantities are shown in Tables.

C. O. B.

Columbus, Ohio, Improved Water- and Sewage-Works.

J. H. Gregory.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 2-119.)

Waterworks.—These works, commenced in 1904, included an improved water-supply for the city and the purification of the dry-weather flow of sewage. Previous conditions were far from satisfactory. The supply was chiefly from ground-water sources, augmented at times by drawing directly from river-intakes, and the danger from typhoid increased until a severe outbreak in 1904 made an improvement necessary. In periods of extreme dry weather practically the whole flow of the Scioto River above the city was taken for water-supply and returned to the river as sewage, which caused much complaint. The population of Columbus in 1907 was estimated at 170,000 and the area within the city limits is 16·81 square miles. The new waterworks begin with a dam 1,006 feet long situated up the river which raises the level by 30 feet and forms a reservoir impounding 1,430,000,000 gallons. Provision is made for raising the dam so as to increase the amount of water impounded to 4,720,000,000 gallons. From the dam the water overflows down the river to a pumping-station 4·5 miles distant, where it is purified and sent through pipes to the city. In dry weather, when the flow over the apron of the dam is not sufficient, gates provide means for adding to the discharge. The dam is curved upstream, but no allowance is made in the calculations for the effect of the arch. The overflow is 500 feet long. The dam is built of concrete, with large stones bedded in it, the total cost of the reservoir and dam being £101,000. The pumping-station and purification-works are designed for an ultimate capacity of 33,200,000 gallons in 24 hours, the present capacity being half that quantity. The raw-water from the river is drawn in by low-lift pumps, and after being filtered, is pumped to the city by triple-expansion steam-pumps with a capacity of 5,800,000 gallons each. The low-lift pumps consist of single-suction, horizontal, Worthington, volute pumps with 26-inch suction and discharge nozzles, direct connected to 12½ by 30 by 17-inch horizontal, tandem-compound, condensing-engines. Each pump has a normal capacity of 16,600,000 gallons per 24 hours.

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[1] *See* previous Abstract.

Sewage-Works.—After experiments costing £9,500 it was decided to construct purification-works to which the whole dry-weather flow of the city is brought by means of a pumping-station. After treatment the effluent passes off into the Scioto River. The pumping-machinery in the main station consists of five units, each being a single-suction, horizontal, volute pump, direct-connected with a vertical, cross-compound engine. Three of the pumps have 20-inch suction and discharge pipes, with a normal capacity of 9,130,000 gallons per 24 hours, while the other two are smaller. The purification-works comprise septic tanks, sprinkling-filters and settling-basins, the present installation having a net normal capacity of 1,660,000 gallons. There are six septic tanks of concrete, each primary tank having a capacity of 590,000 gallons and each secondary tank 2,150,000 gallons. There are four sprinkling-filters radiating from the gate-house, each draining a net area of 2·5 acres. Numerous photographs and drawings accompany the Paper, and the results of the operation of the plant are set forth in Tables.[1]

R. S. B.

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[1] *See also* Minutes of Proceedings Inst. C.E., vol. clxxi, p. 497.

Use of Sulphate of Alumina and Hypochlorate of Lime in the Storage- and Distributing-Reservoir of the Nashville Waterworks. G. REYER.

(Engineering News, New York, 1910, vol. 63, pp. 390–1.)

The successive application and the quantities of the two ingredients, the method used, their cost and the result in eliminating bacteria, are explained, including analyses of the raw and of the treated water. Immediately following are articles on "A 20,000,000-gallon Hypochlorate Water-Disinfecting Plant at Minneapolis, Minn.," by J. A. Jensen, "The Use of Hypochlorate of Lime to Disinfect the Water-supply of Montreal, P.Q.," "The Use of Hypochlorite of Lime in connection with the Mechanical-Filtration Plant of Harrisburg, Pa.," and "Experiments with Hypochlorate of Lime as a Water-Disinfectant at Hartford, Conn.," by E. M. Peck.[1] The first of these additional communications gives facts leading up to the adoption of the plant, which is described with plans accompanied by analytical results. The second gives somewhat similar information, and adds mortality tables and cost per million gallons treated, there being many illustrations. The Harrisburg article contains much detail of the bacterial condition of the water before and after treatment; while the final article gives two Tables showing the results of experimental investigation on the same lines. A leading article[2] sums up and compares the efficiency of the methods described.

C. O. B.

Removal of Iron from Subsoil-Water. DR. FR. BAMBERG.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxxiii, pp. 147–51 *et seq.*)

Iron is for the most part present in underground water in the form of the double carbonate of the protoxide of iron. Only rarely is it found combined with organic matters in the nature of acids and chiefly humic acid; still more seldom as the phosphate, and rarest of all in association with mineral acids, mainly sulphuric acid. Waters holding iron in solution are generally derived from the deeper layers of the soil, in which iron may be contained in the form of pyrites. Water in the upper layers, rich in carbonic acid, penetrates to greater depths and by means of the acid dissolves various mineral substances, and if it encounters iron pyrites it splits it up and forms the double carbonate above mentioned and sulphuretted hydrogen. Combinations of iron with the humus acids are chiefly found in moorlands in peaty districts, while water rich in iron combined with mineral acids is prevalent in the vicinity of lignite deposits. Neither of these latter forms of iron compounds can be got rid of by means of aeration. The methods in which waters impregnated with iron render themselves objectionable for drinking and domestic purposes are discussed, and it is shown that for many industrial uses the employment of such water is quite impracticable. Very frequently, such waters produce various descriptions of *algæ*, which, by their vigorous growth, tend to clog the mains and to give rise to scale and incrustation. The mode by which the oxygen of the atmosphere is made use of to effect the elimination of the iron is well understood and may be considered under two phases:—Firstly, the dissolved iron, in the presence of oxygen, takes the hydrosol form, by colloidal action, and occasions an appearance of turbidity in the water. Then, with more oxygen, it assumes the condition of a hydrogel, causing the well-known brown flakes of hydroxide, which can readily be removed by filtration. Various other dissolved salts and substances play a very prominent part in these operations, and may be divided into two groups—those which favour processes for the removal of iron and those which hinder them. The Author considers and classifies the various matters found dissolved in the usual kinds of water regarded from these aspects. Free carbonic-acid gas and organic substances are the most powerful reagents among the opponents to the aeration process. Full explanations are given of the various systems employed for exposing iron waters to the atmosphere and of the allied processes, together with the results of experiments on a practical scale.

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[1] *Engineering News*, 1910, vol. 63, pp. 391–5.

[2] *Ibid*, vol. 63, p. 406.

G. R. R.

Sterilization of Large Quantities of Water by Ultra-Violet Rays.

V. Henri, A. Helbronner and M. De Recklinghausen.

(Comptes Rendus de l'Académie des Sciences, Paris, 1910, vol. cl, pp. 932–4.)

It was found that a Cooper-Hewitt mercury-vapour lamp was able to sterilize completely an emulsion consisting of typhoid, cholera and pneumonia bacilli at a distance of 60 centimetres in 30 seconds, at a distance of 40 centimetres in 15 seconds, at a distance of 20 centimetres in 4 seconds, and at a distance of 10 centimetres in less than 1 second. An apparatus was then constructed capable of passing under the influence of four such lamps 125 cubic metres of water per hour. When the flow of water was 36 cubic metres per hour it was under the active influence of each lamp for 8 seconds, and therefore, in accordance with the data given above, two lamps should suffice to effect complete sterilization. The following is the result of one of the trials:—The water before it reached the first lamp contained 5,250 germs per cubic centimetre; after passing this lamp it contained 3,650 germs per cubic centimetre, and after passing the second lamp it contained no germs. Each lamp required 660 watts per hour, and these experiments therefore show that large quantities of water can be completely sterilized by means of the ultra-violet rays produced by the Cooper-Hewitt mercury-vapour lamp for an expenditure of 36 watt-hours of electricity per cubic metre.

H. R. S.

Intensive Illumination by Means of Gas.

(La Nature, Paris, 1910, Deuxième Semestre, pp. 69-73.)

There is a growing demand for more powerful methods of lighting. For a time the electric arc-lamp proved superior to any form of illumination for large spaces, though the Auer mantle enabled the gas-jet to rival the electric glow-lamp for certain purposes. Recently it has become possible, by means of apparatus differing considerably from the principle of the Bunsen burner with the mantle, to utilize gas for the production of intense illumination. In the first instance, attempts were made by grouping a number of burners provided with mantles in one lamp to obtain a powerful light, but this proved costly and inadequate. To obtain an intense light it becomes necessary to decrease the volume of the flame and to augment the supply of air injected with the gas. This has been accomplished in two different ways, either by employing the burner itself to supply the requisite power, or by calling in the aid of some external source of power. The former plan, apparently the more simple, rational and economical, is used in the Lucas, the Scott-Snell and the Soleil d'Or lamps, which rely either on the automatic compression of the air-supply or the creation of a forced draught. Both the new Auer and the Kern lamps are adapted to give a high-power light by forced draught. Some of the latest types of intensive burners require the aid of a compressor. In the Keith lamp this compressor is operated by a small motor which may be driven by electricity, hot air, petrol or hydraulic pressure. It is possible with this lamp to obtain a 1,500-candle-power light. Details are given of the Pharos system, as used at Stuttgart, for street lighting. The Selas system differs from the foregoing, owing to the fact that a suitable mixture of air and gas is compressed, lamps up to 5,000-candle-power on this principle being in use in Berlin for lighting some of the principal thoroughfares.

G. R. R.

Rare Earths and Incandescent-Gas Lighting.

Captain NICOLARDOT.

(Bulletin de la Société d'Encouragement pour l'Industrie Nationale, Paris, 1910, vol. 113, pp. 436–84.)

Reference is made to the work of Crookes, and to his view that each band in the spectrum of the rare earths is due to the presence of some single element. While many new elements have been isolated, it is still a matter of extreme difficulty to determine whether any particular oxide represents a single substance or a group of combined oxides. Dealing with the history of these rare earths the Author starts with the discovery of the so-called tungsten by Cronstedt in 1750, and notices the researches of Hisinger, Berzelius and others, and the separation of yttria and thorium. A graphic diagram shows the various groups of allied elements. Passing on to the employment of these earths for illuminating purposes the Author refers to Auer's investigations about 1880, and to the results obtained by him with the oxide of lanthanum and subsequently with zirconium and thorium. Details are given of Auer's first patent secured in 1885, together with the modification arising from the presence of minute quantities (1 per cent.) of cerium nitrate, which really proved the clue to success in incandescent-gas lighting. The preparation and uses of monazite sands are discussed with photographs of the washing processes in North and South Carolina and in various parts of Brazil. A Table shows the great fluctuations in the value of thorium nitrate obtained from this sand, prices varying between £100 per kilogram in 1894 and £1 per kilogram in 1910, and it is stated that these values are solely due to the existence of a cartel. The different modes of treating the monazite sand are explained and an account is given of the manufacture of mantles in all stages, from the knitting of the fabric to the packing of the goods for the market. Machine processes of manufacture have now entirely superseded hand work, the machine of Robin, which is said to be based on that of Müller and Bonnet, making 6,000 mantles per day. Various processes of obtaining very high illuminating power by the use of compressed-air and compressed-gas are described, and details are given of the lamps and burners employed.

G. R. R.

High-Pressure Submarine Gas-Main. R. L. CLARKE.

(American Gas Light Journal, New York, 1910, vol. xcii, pp. 60–1.)

The towns of San Diego (California) and Coronado are situated on opposite banks of a bay 4,000 feet wide, and for the purpose of conveying to the latter town gas generated at the San Diego Works a 4-inch heavy cast-iron pipe, made in lengths of

12 feet, was laid under the water. Flexible ball-and-socket joints were made at distances of 24 feet in the pipe-line, the intermediate joints being standard socket and spigot. Five hundred feet of pipe could be laid at low tides, whilst the balance was laid from a chute built up from one end of a lighter 24 feet wide by 90 feet long. The lighter was moved across the bay by the use of windlasses attached to anchors. The chute consisted of two 80-foot piles bolted together and erected on framework to assume an angle of about 50° with the horizontal. The pipes were jointed on the chute and lowered by moving the lighter. Two 100-foot piles placed 4 inches apart and bolted together trailed from the lighter, and by means of iron trucks fixed to their lower ends rolled on the bed of the bay. The pipes were thus protected, while being lowered, against the wind and tide. It required three days to lay the pipe across the bay and eight days for divers to caulk the pipe. By testing with 75 lbs. air-pressure, a loss of 10 lbs. pressure was realized in 12 hours.

E. V. E.

Manufacture of Benzol. A. GREBEL.

(Le Génie Civil, Paris, 1910, vol. lvi, pp. 381–6.)

Twenty years ago the gas companies were the sole producers of benzol, and, as the illuminating value of coal-gas is greatly enhanced by its presence, they naturally took no steps to remove that contained in the gas itself, but confined their attention to the recovery of the very small proportion contained in the tar, a quantity which amounts only to about 1 lb. for every ton of coal carbonized. The gas derived from coking ovens is itself a by-product, and its principal use being to generate the heat required in the carbonizing of the coal, any admixture of benzol is practically without value, and this substance is now recovered by the following process which yields about 10 lbs. of benzol for every ton of coal carbonized:—The gases are passed through a series of scrubbers by which they are brought into intimate contact with heavy tar-oil in which the benzol is soluble. The circulation of the oil through the scrubbers takes place in a direction opposite to that of the gases, thus when the oil is fresh it comes into contact with gases more or less depleted of benzol, while when it has become saturated and has so lost its avidity for absorption the gases which it encounters are rich in benzol. The temperature of the oil should not exceed 25° C. when, under suitable conditions, a given quantity of oil will deal with about a thousand times its volume of gas. Two forms of scrubber are described, the Zschokke and the Mallet; in the former the oil trickles through a series of wooden grids one above the other, while in the latter several similar diaphragms are superposed, and by a multitude of trapped holes in each the ascending gases are made to bubble through the oil. The latter loaded with benzol is then heated by steam, when it parts with what it has absorbed, and after having been cooled down is again ready for passing through the scrubbers. The benzol driven off, after having been rectified, is treated with concentrated sulphuric acid and subsequently with soda, after which it is again rectified and is then put on the market as 90 per cent. benzol. This mixture, which contains about 84 per cent. of benzene C_6H_6 and about 15 per cent. of toluene C_7H_8, begins to boil at 81° C. and is suitable for use with the internal-combustion engine.

A description is given of a plant, recently erected near Douai, capable of dealing with 7¾ million cubic feet of gas per day.

I. C. B.

Horizontal Carbonizing-Chambers at Innsbruck. O. PEISCHER.

(Journal für Gasbeleuchtung und Wasserversorgung, Munich, 1910, vol. liii, pp. 219–21.)

The carbonizing-chambers or ovens at Innsbruck are sufficient for the production of all the gas required for the supply of the town, and for 3 months the gas supplied has been manufactured exclusively by this method of coal-carbonization. The installation is built on the Koppers system and consists of four beds of three chambers. A charge of about 75 cwt. is introduced into each chamber, and is, in most cases, subjected to carbonization for a period of 24 hours. The maximum daily production of gas was 525,134 cubic feet; the minimum 261,331 cubic feet. From a trial extending over 11 days it was found that 1 ton of Saar coal yielded 11,787 cubic feet of gas (corrected to 30 inches barometer and 60° F.) of an average calorific power of 633 B.Th.U. per cubic foot. The coke used for fuel (expressed as dry coke and free from ash) amounted to 12 per cent. by weight of the dry coal carbonized. The average working results obtained over the whole working period are also given:—(1) Yield of gas per ton of air-dried coal (as charged) 10,985 cubic feet, corrected to 30 inches barometer and 60° F. (2) Calorific power of the gas, 644 B.Th.U. per cubic foot. (3) The coke obtained represented 70 per cent. by weight of coal carbonized and contained 5·5 per cent. of breeze and slack. (4) A thin tar representing 5·6 per cent. by weight of coal carbonized was obtained. (5) The yield of ammonia was equal to 0·22 per cent. by weight of the coal carbonized. It is pointed out that the plant did not work continuously at its full capacity, and that this would account for the later series of results comparing unfavourably with that of the special test. The cost of the installation was £14,875, including foundations, flues and chimney, gas-delivery pipe to the end of the setting, coal-handling plant and coke-gantry. By accepting 618,000 cubic feet as the daily consumption, a reserve of one-sixth is allowed, and with this figure the cost of production is equal to £24 1*s.* 5*d.* per 1,000 cubic feet. Under similar conditions the cost of inclined-retort working is £21 12*s.* 6*d.*, of vertical-retorts, £31 9*s.* 7*d.*, and of inclined carbonizing-ovens, £40 2*s.* 5*d.*

E. V. E.

Quenching and Transporting Gas-Coke. FEILITZSCH.

(Journal für Gasbeleuchtung und Wasserversorgung, Munich, 1909, vol. lii, pp. 1119-21.)

In this installation of the Illig process of quenching and transporting gas-coke, a perforated iron vessel, 11 feet 10 inches long and 2 feet 2½ inches high, is sufficiently capacious to hold the coke discharged from three retorts. This vessel is lowered into a tank 16 feet 5 inches long and 3 feet high, half filled with water, which may be moved by an endless chain driven by an electromotor to any suitable position in front of the discharging end of the retort-bench. The perforated vessel containing the charge of coke, after having been lowered by means of overhead gear into the water-tank, is raised slightly to allow for draining and then transported to the storage-ground. By the careful adjustment of working conditions, the coke may be dry by the time it is lowered on to the coke-heap. It has been found that by falling about 5 feet from the transporter to the storage-ground, the material consists as follows:—

89·6 per cent. of coke	¾ inch to 11¾ inches in size.
6·6 ,, ,, ,, ...	$\frac{3}{16}$ inch to ¾ inch in size.
3·8 ,, ,, breeze . . .	Up to $\frac{3}{16}$ inch in size.

The quantity of water required for quenching is 134 gallons per ton of coke produced, the water being raised in temperature from 57° F. to between 167° and 176° F.

As regards the power required, one electromotor serves for driving a De Brouwer charging- and discharging-machine, together with the coke-transporter 230 feet long, and the endless-chain gear for adjusting the position of the water-tank. A total of 0·468 Board of Trade unit is required, and is comprised as follows:—

Charging- and discharging-machine	0·310 unit.
Electric transporter	0·118 ,,
Quenching-tank	0·040 ,,

With the price of electric power at 0·72*d.* per unit this total represents 1⅓*d.* per ton of coal carbonized. The installation costs little and is suitable for small as well as large works. The Paper is illustrated.

E. V. E.

Trial with Vertical Retorts at Mariendorf. E. KÖRTING.

(Journal für Gasbeleuchtung und Wasserversorgung, Munich, 1910, vol. liii, pp. 4–6.)

The setting under observation, built at the Mariendorf Gasworks at Berlin, is of the Dessau type, but contains eighteen retorts which are built over one producer. It has been customary to allocate twelve retorts to each setting, and in that case six retorts have been built in two parallel rows. With the increased number of retorts, three rows of six retorts are built into each setting, and no more ground space is occupied than with the former setting of twelve retorts. The retort-bench had been in use for some time prior to the commencement of this trial in November 1909. The coke burnt in the producer was of the same kind throughout the experimental period and was produced from the carbonization of a mixture of two-thirds English coal (Boldon) and one-third Silesian coal. It contained 12·4 per cent. of ash and possessed an average calorific value of 12,573 B.Th.U. per pound. During the trial a Silesian coal termed "Concordia" was carbonized. The retorts were scurfed 8 days prior to the trial. It having been previously determined that the most favourable results were to be obtained by carbonizing for 10½ hours with the admission of steam during the last 2 hours, these conditions were adopted, and the trial consisted of five periods of 21 hours' working. The averaged results, given in a Table, show the make of gas per ton of coal carbonized to be 13,206 cubic feet (corrected to 30 inches barometer and 60° F.). The gas yielded an average calorific power of 543·6 B.Th.U. per cubic foot, and an average illuminating power of ten candles, when tested in the Carpenter burner. The specific gravity was 0·445, and the average contents of nitrogen 1·2 per cent. by volume, no air having been admitted during oxide purification.

The following Table compares the results obtained during this trial with those obtained when using a similar type of setting but

	Setting of 18 Retorts.	Setting of 12 Retorts.
Per twenty-four hours—		
Coal carbonized . . . . Cwt.	390	273
Gas produced . . . . Cubic feet	258,323	186,874
Coke used as fuel . . . . Cwt.	46·3	38·75
Per ton of coal carbonized—		
Gas produced . . . . Cubic feet	13,206	13,616
Calorific power of gas . . B.Th.U. per cubic foot	543·6	537·6
Coke used as fuel . . . . Lbs.	26·4	31·5
Coke used as fuel—		
Per 1,000 cubic feet of gas made . . . . Lbs.	21	25
Heating value of coke used per 100,000 B.Th.U. realized in gas manufactured	45,700	53,400

one into which twelve retorts were built. Precautions were taken to operate under similar conditions. The increased efficiency of the eighteen-retort setting is due to the reduction of the quantity of fuel required for heating purposes.

E. V. E.

Three-Phase Arc with Four Carbons. A. Righi.

(La Lumière Electrique, Paris, 1910, vol. 9, pp. 41–3.)

After reviewing the work of Mercanton, Bentivoglio, Siciliani and Fleming, the Author describes an arc produced by him consisting, as regards the lower electrode, of three carbons each 8 millimetres in diameter placed in a triangle of 12 to 15 millimetres side, and, as regards the upper electrode, of a single carbon of 20 millimetres diameter, this fourth carbon being usually insulated. Photographs of the arcs produced with and without the fourth carbon are given. The results of tests hitherto obtained show that a close approach of the upper to the lower electrode doubles the number of craters, reduces the watts absorbed, increases the illumination and steadies the light. It appears possible to run the arc on as low a frequency as fifteen cycles per second, rendering it suitable for lighting railway-stations, etc., supplied by low-frequency currents for power. The upper carbon has a very small consumption.

J. A. S.

Calculation of a Coil-Winding. Eugene H. Weiss.

(La Lumière Electrique, Paris, 1910, vol. 9, pp. 42–4.)

The Author deals with the problem of obtaining a given resistance of a coil, given the dimensions of the bobbin, the diameter, depth of insulation and resistance of the wire. If the depth of insulation be less than one-fifth the radius of the wire, the formula

$$r = 4\sqrt{\frac{l\rho\,(\mathrm{R}'^2 - \mathrm{R}^2)}{\pi\,\Omega\,\mathrm{K}}}$$

is sufficient, where r = radius of the bare wire, l = length between bobbin cheeks, ρ = specific resistance of wire, R′ = overall radius of bobbin, R = radius of bobbin core (external), Ω = resistance to be obtained, and K is a coefficient greater than unity depending on the space between turns. Calculations for K are given in the cases where: (1) the turns of the layer rest midway between the turns of the layer beneath (K = 1·10); (2) where each turn lies immediately above the corresponding turn of the layer beneath (K = 1·27);

and (3) where insulating-strips are placed between each layer, where

$$K = 1\cdot 27\left(1 + \frac{\text{thickness of insulating-strip}}{\text{apparent diameter of wire}}\right).$$

J. A. S.

Commutation. F. W. Carter.

(Electrical World, New York, 1910, vol. lv, pp. 804-7.)

The Author discusses the resistance and reactance of coil-circuits during the commutation-period, investigating the phenomena occurring near the end of the period. Mathematical treatment is given to the cases (1) in which one coil only is short-circuited, (2) in which two coils are simultaneously short-circuited, (3) in which the coils considered are short-circuited by brushes of opposite polarity provided the short-circuits of the two coils be not opened at the same instant, and (4) the general case presented by an average commutating-machine in which there may be many studs of brushes each short-circuiting two or more coils, capable of affecting one another inductively. In the last case it is shown that to obtain naturally sparkless commutation $\frac{RT}{\Lambda} > 1$, where $T =$ total time of short-circuit, R is the contact-resistance of the brushes, and Λ a value whose first approximation is twice the slot-inductance. R is a function of the time and is thus variable. The provision of resistance-leads between coils and commutator does not tend to satisfy the above inequality. If, near the end of the commutation-period, the area of contact on the toe-segment diminish faster than the rate of time-lapse, the tendency to infinite current-density, or sparking commutation, disappears, whatever the value of $\frac{RT}{\Lambda}$. Hence serrated contact-surfaces on the toe of the brushes are recommended. The Author then discusses sparkless commutation by commutating-poles, time-values of current and flux being given in a particular case.

J. A. S.

Aluminium as a Conductor. J. C. Smith.

(Transactions of the Canadian Society of Civil Engineers, Montreal, 1909, vol. xxiii, pp. 366-78.)

Notwithstanding various objections to the use of aluminium as a conductor, such as its high coefficient of expansion, the difficulty in making joints, the highly electro-positive nature of the metal, and its low melting-point, it has steadily advanced in favour, and at present more than half the high-tension transmission-lines which are being erected are using this metal. At present copper is selling at about 7·25*d.* per lb., while aluminium wire can be purchased for about 10*d.* per lb. Engineers agree that no transmission-line should be smaller than 0·04 square inch of copper, and for aluminium this would have to be increased, for mechanical reasons, to about 0·06 or 0·08 square inch. Aluminium cables for the same conductivity have a diameter 29 per cent. greater than copper, and therefore a larger area exposed to wind and sleet. The high coefficient of expansion requires greater sag in stringing the aluminium wire, and almost all the trouble due to the use of this metal has arisen from lack of care in this particular. The Author, after an experience of two years in the use of this metal on high-tension lines, finds that a properly designed aluminium line does not give any more trouble than one of copper. In the case of a transmission system comprising 400 miles of high-tension aluminium line the results have been satisfactory, and some of these lines have been working for 6 years.

R. S. B.

Coupled Circuits in which the Secondary has Distributed Inductance and Capacity. LOUIS COHEN.

(Bulletin of the Bureau of Standards, Washington, 1909, vol. vi, pp. 247-53.)

In a previous communication[1] the Author has shown that when the inductance and capacity of two circuits are localized the system oscillates with two distinct frequencies. In wireless telegraphy, however, the capacity and inductance are distributed, and consequently the system will oscillate with an infinite number of frequencies, and it is such a system which the Author investigates in this communication. He states the fundamental equations for both circuits, and from these he derives a transcendental equation, the infinite number of roots of which are the frequencies required. This equation is solved graphically by finding the intersection of two sets of curves, both when the coefficient of coupling is 0·25 and when it is 0·1. In the latter case the roots have values: 58, 82, 290, 420, 590, etc. The first two roots represent the two fundamental frequencies, and the frequency of the first harmonic is about four times the frequency of the fundamental. If, therefore, the system is in resonance with the fundamental frequency the upper harmonics will have no influence on the working of the system.

H. R. S.

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[1] Minutes of Proceedings Inst. C.E., vol. clxxix, p. 456.

Disruptive Strength with Transient Voltages.

J. L. R. HAYDEN and C. P. STEINMETZ.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 747–76.)

Professor Creighton has expressed the opinion that even in air there is a time-lag in disruptive discharges. This matter is of industrial importance, and the Authors describe an apparatus by means of which this time-lag can be measured both in air and in oil. A continuous rising voltage was impressed on the primary coil of a high-voltage transformer, thus inducing a voltage-impulse in the secondary of the transformer. It is shown in an Appendix that the duration of the transient voltage thus produced increases with the current and inductance, and decreases with the voltage. In the experiments referred to the voltage in the primary was 50 to 350 volts and the current 0·25 to 4 amperes; the ratio of transformation was 1–300, and the duration of the transient voltage was 0·00014 to 0·0158 second. Needle-points and spheres were used for the electrodes, and the results of the experiments are given in a tabular form and are also plotted. Some of the principal conclusions arrived at are as follows:—The disruptive discharge through a dielectric requires not merely a sufficiently high voltage, but a definite minimum amount of energy; the disruptive discharge does not occur instantly with the application of voltage before the discharge occurs. The disruptive energy needed for oil is about thirty times greater than that for air. Formulas are derived for calculating this energy under different conditions; that with 3·8-centimetre spheres in dry white paraffin-oil the energy (in joules) is = 10 (distance apart of electrodes) $^{1\cdot5}$.

H. R. S.

Proportioning of Electrodes for Furnaces.[1] CARL HERING.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 285–334.)

Mr. Carl Hering here develops, especially from the engineering point of view, the laws of electrode-losses in arc furnaces he published in a Paper read before the American Electrochemical Society, and gives rules and data for the proper proportioning of the electrodes, the selection of the best material, the calculation of the losses, etc. The fundamental principle on which his analysis is based is that the heat-gradient at the hot end of the electrode should be zero. When this condition obtains no heat from the furnace or its products is lost through the electrode. The investigation results, amongst other things, in determining two new measures of electrode qualities which the Author calls the "electrode voltage" and the "specific cross-section." These quantities are physical properties quite as much as conductivities, specific heat, etc. The former has to do with the power-loss, and the latter with the size, of the electrode; the lower their values the better suited is the material for electrodes. The electrode-voltage is equal to $\sqrt{2 \times \text{thermal conductivity} \times \text{electrical resistivity}}$. The specific cross-section is equal to $\frac{\text{electrical resistivity}}{\sqrt{2 \times \text{thermal conductivity}}}$. The total voltage is obtained by multiplying the electrode-voltage by the square root of the temperature-drop between the hot and cold ends, and the section of the electrodes is obtained by multiplying the specific cross-section by the current, by the length of the electrode and by the square root of the temperature-drop. The Author gives, both in a graphical and in a tabular form, the results of an extended series of trials made with electrodes consisting of carbon, graphite, iron and copper. It was found that the values of the electrode-voltage and of the specific section varied with temperature. A Table shows some of the values obtained, and it is to be observed that those given for a furnace-temperature of 1,400° C. were exterpolated, because the furnace experimented with did not permit of reaching this temperature. The Paper concludes with a discussion of the relative advantages and disadvantages of the various materials tested for use as electrodes, and in every case the carbon electrodes come out badly.

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[1] *See* following Abstract.

Material of Electrode.	Temperature.	Electrode-Voltage.	Specific Cross-Section.
	°C.		
Carbon	751	0·0630	0·0239
	942	0·0639	0·0232
	1,400	0·065	0·022
Graphite	720	0·0454	0·00695
	914	0·0447	0·00722
	1,400	0·042	0·0073
Iron	727	0·0109	0·00254
	912	0·0112	0·00287
	1,245	0·0127	0·00313
	1,400	0·0128	0·0033
Copper	541	0·00596	0·000310
	837	0·00697	0·000381
	1,400	0·0086	0·00050

H. R. S.

Modifications in Hering's Laws of Furnace-Electrodes introduced by including Variations in Electric and Thermal Resistivity.[1] A. E. Kennelly.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 267–83.)

In 1909 Mr. Carl Hering, in a Paper before the American Electrochemical Society, enunciated at least seven important laws of electrode losses. Two of these are now considered to see what effect is produced in them by including variations in the electric and thermal resistivity. The matter is investigated mathematically by determining the fundamental differential equation connecting temperature with distance along the electrodes. Two numerical examples are worked out and are also plotted graphically. If the electric and thermal resistivities were constant the temperature-gradient along the electrode, supposing that the temperature be maintained at the hot end without any current, would be represented by a straight line. When, however, the current is flowing, and both ends are maintained cold, the temperature-gradient would be represented by a parabola with its vertex at the middle point of the electrode. The actual gradient is found by superimposing the parabola on the straight line. The loss of heat is least when the tangent to the resulting curve is horizontal at the hot end of the electrode; that is to say, when the temperature gradient at that point is zero. The resistivities, however, vary with temperature, and in consequence the straight line becomes a slightly concave curve and the parabola becomes a cubic; when superimposed the tangent at the hot end is no longer horizontal. Mr. Hering's first two laws, as amended by Mr. Kennelly, are: (*a*) The combined loss through the cold end of an electrode is equivalent to the sum of the loss by heat conduction alone (when there is no current) and approximately half the I^2 R loss, the exact fraction depending on the temperature-coefficients of the resistivities. With constant resistivities, the fraction would be one-half. (*b*) The above combined loss will be least when the loss by heat-conduction alone is equal to the joulean loss through the cold end. The total loss will then be approximately equal to the total joulean loss and very little heat will flow into or out of the furnace. With constant resistivities, this resultant furnace-flow will be nil.

H. R. S.

Compounded Alternator. J. A. Montpellier.

(L'Électricien, Paris, 1910, pp. 49–51.)

The Author describes a system of compounding alternators devised by Mr. Marius Latour to regulate automatically the pressure of the current from an alternator at all conditions of load and power-factor. The essential feature is the use of a special exciter rotated at the speed and having the same number of poles as the generator. The exciter supplies direct current by the ordinary commutating-device, and also an alternating pressure tapped from the same armature by means of slip-rings. The field of the exciter is provided with interpole fields to keep the distribution of flux constant. The direct current energises the alternator-field in the usual way. The alternating electromotive force is impressed on a circuit leading first through a supplementary winding in the stator and then through the secondary of a transformer whose primary is in one of the phases of the main generator supply. An alternating electromotive force is induced in the supplementary stator-winding, opposing that of the exciter alternating-pressure. Theoretically, when the exciter-characteristic is correct and the preliminary regulation is properly effected, the compounding current in this circuit is nil. Passage of current in the main alternating circuit, however, upsets this relationship, allowing compounding alternating current to flow from the exciter-tappings and strengthening the total impressed volts on the alternator to an extent vectorially equal to the reactance-drop. Where desired the machine can be over-compounded. Vector-diagrams and a proof are given.

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[1] *See* preceding Abstract.

J. A. S.

Tests of Turbine-Alternators. H. N. Nichols.

(Electrical World, New York, 1910, vol. lv, pp. 282–5.)

The Author describes the practice in turbo-alternator testing adopted by one of the large electric manufacturing companies. A description of the appliances for commercial and experimental tests follows, and particulars are given of the insulation-test of armature-coils, circuit resistance (cold), brush-adjustment, turbine-balance, phase-rotation, high-speed tests, open-circuit saturation, synchronous impedance, heating-runs, etc General instructions for the use of testing-staffs are appended.

J. A. S.

Damping-Couple in Alternators. C. F. Guilbert.

(La Lumière Électrique, Paris, 1910, vol. 9, pp. 355–61, and 387–91.)

After reviewing the types of damping-coils, their action is discussed. The work of previous investigators, notably Mr. F. Emde, is discussed and a numerical example is worked out. The Author then takes into account a considerable number of experimental factors, such as the stray fields, in order to calculate very exactly the value of the couple. The action of the damping-coils is at a

maximum at the condition of resonance, or in the normal condition of operation, and is due to the flux arising from the induced, and not the inducing, currents. The subject is treated mathematically by the formula of Mr. Boucherot.

J. A. S.

Use of Vector Components in Calculating Damping Effects on Alternate-Current Machines. J. A. Schouten.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 282-285.)

The Author bases his mathematical deduction on the well-known law that the sum of the component vectors of any effect is equal to the sum of the vectors. He first considers the magneto-motive force of a 3-phase alternate-current magnet. Starting with the analysis of the sine-curves having only one phase and one winding, he develops the method for 3-phase currents and explains minutely why the higher sine-values may be neglected in the expression:

$$1 = \frac{4}{\pi} (\sin x + \tfrac{1}{3} \sin 3x + \dots)$$

thus simplifying to $1 = \frac{4}{\pi} \sin x.$

The important component opposing the magneto-motive force of the field-winding is that component moving with the same velocity as the armature.

The nth harmonic magneto-motive force may be represented by a sine-curve showing $2n$ tops for each part of the stator's contour, on which the harmonic has 2. Its amplitude is $\frac{1}{n}$ of that of the first, its velocity $\left(\frac{1}{n} \times s\right)$ of that of the armature, where s denotes the velocity of the sychrone. The velocity relative to the magnetic field is $s\left(1 \pm \frac{1}{n}\right)$.

For a 3-phase machine, in which the third harmonic drops out, the fifth, seventh, eleventh, etc., field would increase the electro-motive force of the opposed field by:

$\frac{1}{25} = 4$ per cent., $1\frac{1}{49} = 2$ per cent., $\frac{1}{121} = 0{\cdot}8$ per cent., etc.

This shows that, but for the damping effect, the higher harmonics could not be neglected in the above expansion. Working out the damping effect the Author obtains the following expression for the speed of the frequency of the eddy-currents set up in the poles:

$$n \sim \left(1 \pm \frac{1}{n}\right) = n \sim \pm 1$$

where $\sim$ denotes the fundamental frequency. This speed is for the fifth harmonic at fifty periods equal to 249, and for the seventh harmonic 357, etc. These large frequencies will generate enormous eddy-currents, even with a weak field, thus damping the field effect, i.e., magneto-motive forces are generated, which practically neutralize the effect of the existing magneto-motive force. For unsaturated iron it may therefore be said that the higher harmonic magneto-motive forces are neutralized, i.e., the higher fields have been damped.

H. C. A. T.

Tension-Regulation of Alternating-Current Machines.

Dr. W. LULOFS, M.Sc.

(De Ingenieur, The Hague, 1910, vol. xxv, pp 378–81.)

The influence of the load on the tension-regulation of electric machines is much smaller and simpler for direct- than for alternating-current machines. Direct-current machines can now be made with the brush in the neutral zone, whereas with alternating-current machines the angle between the armature-field and the excitation-field is determined by the difference in phase between the current and the electromotive force. This forms the principal difference between the tension-regulation of direct-current and alternating-current machines. If the specified tension-increase from "no load" to "full load" is small the tension-regulation is easily effected. This increase in tension depends on (1) armature-field, (2) demagnetization of the armature, (3) increased leakage-field of the poles, and (4) saturation-factor: 1 and 3 can be reduced to a minimum by suitable constructional methods, and 2 is fulfilled by having a fairly high short-circuiting current as compared with the maximum load-current.

These reasons explain why it is only possible to obtain an equally good tension-regulation for turbo-generators as for slow-speed generators when the load on the armature is reduced to a minimum, which reduction would entail an increase in the price of the machine per kilowatt-capacity. This increase can only be kept down by sacrificing the efficiency of the tension-regulation. The following Table bears out these statements:—

	Drop in Tension at Full Load.	Value of cos ϕ.
	Per Cent.	
Turbo-generators	5–7	1·0
	13–15·5	0·9
	15–18	0·8
Generators actuated by slow-speed engines	3–4	1·0
	7–9	0·9
	11–12·5	0·8

The enormous influence of the demagnetizing armature-ampere-windings renders it highly desirable to establish a method by which this influence may be determined on a finished machine. Several methods for determining this influence have been suggested; that of Mr. Gisbert Kapp is adopted by the Author, who gives a mathematical analysis of it.

H. C. A. T.

Poynting's Formula and the Energy-Current in Dynamos.

COMTE DE BAILLEHACHE.

(La Lumière Électrique, Paris, 1910, vol. 9, pp. 166–71 and 199–208.)

Poynting's formula, connecting the quantity of energy traversing a unit of surface in a unit of time with the intensities of the electric and magnetic fields, is first developed mathematically, Poynting's conception being modernized by reference to the conception of Lorentz of a system of ponderable bodies containing ions, whose movements engender an electric force of field-intensity, the total of which is $E = E_1 + E_2 = 4\pi V^2 \delta + v H$, where δ = dielectric displacement, V = velocity of light or the relation of electrostatic and electromagnetic units, v = velocity of the ponderable material, H = magnetic force, E_1 = electrostatic force, E_2 = the vector product of v and H. Then S, the density of energy current $= \frac{1}{4\pi}(E\,H)\,n$. As this is a result of vectorial quantities the subject of electric eddies is next treated, divided into a consideration of surface-eddies and space-eddies, and the vectorial diagram is applied. Poynting's vectorial quantity S is then applied to the study of the transformation of energy in dynamos and alternators.

J. A. S.

Induction-Generator. THOMAS SPOONER and A. J. BARNES.

(Electrical World, New York, 1910, vol. lv, pp. 464–5.)

As a New York Company is about to use two large induction-generators for power-station work, the Authors have investigated characteristics bearing on the operation of induction-generators in parallel with synchronous apparatus in a railway generating-station. The first point investigated is the effect of short circuit on the system, curves being taken on a modified double-vibrator Blondel oscillograph, which show that a short circuit is equivalent to withdrawing exciting current from the induction-generator, causing it to cease delivering energy. Oscillograph-curves, which are given, were taken on short circuit when the induction-generator was supplying a non-inductive and a capacity load. Another point investigated was the instantaneous value of the current on connecting the induction-generator to the circuit at various speeds above and below synchronism, including standstill, and simultaneous records of voltage and current on the generator upon switching in were taken. The Authors arrive at the conclusion that such generators are particularly useful in connection with exhaust-steam turbines run in connection with reciprocating-plant driving synchronous generators. As short circuits frequently occur in railway practice such machines are especially suitable for railway power-houses.

J. A. S.

Mutual Induction between Phases. J. REZELMAN.

(La Lumière Électrique, Paris, 1910, vol. 9, pp. 99–105.)

The Author gives the details of calculations and tests made on the stator of a three-phase turbo-alternator of 4,000-kilovolt-ampere capacity with four poles to run at 1,500 revolutions per minute and giving 6,000 volts at 50 periods. The tests were made with the rotor removed. Diagrams of various methods of winding are given, together with drawings of the end-connections and disposition in the slots. The subject of mutual induction is first treated mathematically, the results being checked by actual tests.

J. A. S.

Routin Automatic Regulators, applicable to Alternators, Turbines, etc. ALIAMET.

(L'Électricien, Paris, 1910, pp. 129–33.)

The Author points out that with compounded alternators the correction of pressure-variation depends on constancy of speed, and that frequency-variation in all alternators depends on variation of speed. Devices, such as direct-compounding, automatic regulation of pressure or speed, have so far not been entirely successful. The Routin regulator comprises an electric measuring-instrument (voltmeter, ammeter or wattmeter), furnished with maximum and minimum contacts, and a system of relays operating an auxiliary motor which takes charge of the regulating movements. Among the applications of the device are the maintenance at constant voltage of the further end of a feeding-cable, the speed-regulation of steam- or water-power engines, the regulation of electric furnaces, etc.

J. A. S.

Consumption of Lubricants in Turbo-Generating-Stations.

Dr. A. C. VAN ROSSEM.

(De Ingenieur, The Hague, 1910, vol. xxv, p. 361.)

The Author compares by a Table the annual cost of lubricants for generating-stations with reciprocating-engines and for those with turbines, much to the advantage of the latter.

Year.	Reciprocating-Engine. Cost per Kilowatt-Hour.	Turbines. Cost per Kilowatt-Hour.
	d.	*d.*
1907	0·048	0·0026
1908	0·042	0·0026
1909	0·030	0·0012

The cost of lubricants used for reciprocating-engines amounts to 6 to 10 per cent. of the coal bill, and may, under favourable circumstances, be reduced to 4 per cent. With turbines, however, this cost is reduced to ½ to 1 per cent. The Author found that the percentage of tarry substance contained in the oil increased as the oil was used over and over again. This percentage was, after 2,400, 4,600 and 10,000 working-hours, 0·4, 0·6 and 0·8 respectively. The viscosity was reduced from 5·3 to 4·7 after 10,000 working-hours, the temperature of the oil being about 122° F.

H. C. A. T.

Insulators for High-Tension Circuit. K. KUHLMANN.

(Elektrotechnische Zeitschrift, Berlin, 1910, vol. xxxi, pp. 51–5, 86–90, 116–9.)

The subject of porcelain insulators for high voltages is fully discussed and many kinds are described and illustrated. A curve shows what the flash-over voltage should be for any given line-voltage. The ratio of these two voltages follows sensibly a hyperbolic curve; at 1,000 volts line-voltage the ratio is 6·5 and at 100,000 volts 2·3. The flash-over voltage is supposed to be taken when the insulator is exposed to rain falling at the rate of 10 millimetres per minute. This rainfall figure is based on curves, giving the relation between the flash-over voltage of various insulators and the amount of rainfall, which show that for greater rainfalls than 10 millimetres per minute there is very little diminution of the flash-over voltage. The Author shows that the best results will be obtained when the electric capacity of the insulator is least, and it follows from this that the petticoats should be well spread. Tables of sizes, weights, flash-over distance and flash-over voltage when dry, when covered with graphite dust and when exposed to 10 millimetres per minute rainfall, are given of "Delta" and of "Rosenthal" porcelain insulators, and it is shown that such insulators become too heavy if the line-voltage exceeds 25,000. There are several photographic reproductions of insulators exposed to flash-over voltage, both when dry and in the rain.

H. R. S.

Practical Method of Protecting Insulators from Lightning and Power-Arc Effects. L. C. NICHOLSON.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 241-66.)

The complete protection of high-tension porcelain insulators from destruction by lightning effects has not yet been completely attained, notwithstanding numerous devices adopted for the purpose. The 60,000-volt transmission-lines of the Ontario Power Company, which were put into operation in July, 1906, have suffered greatly from such effects, and an account is given of the troubles experienced and of the different measures which have been adopted to minimise them, resulting finally in practical success. The original porcelain insulator adopted consisted of three parts, or petticoats, and each part was tested with 75,000 volts for 3 minutes, but the complete insulator was not tested after assembling. The flash-over voltage of the complete insulator, when dry and clean, should have been 195,000 volts, and when exposed to $\frac{1}{5}$ inch per minute precipitation at 45° it should be 120,000 volts. A photograph of twenty of these insulators which were damaged during a single storm in 1906 is given, and in almost every case the lower petticoat suffered most. It was definitely observed that a lightning stroke 500 feet distant may cause a flash-over or a puncture. Each pole carried three insulators, and it was found that the number of insulators disabled on the top wire was four times greater than on a single side wire. In 1907 an attempt was made to improve matters by inserting relief-gaps at regular intervals on the top wire, but no beneficial result was obtained. The lightning season of 1908 was particularly severe, 226 insulators being destroyed, of which 114 were punctured, 31 were shattered by direct stroke and 81 were shattered by the power-arc following the flash-over. To overcome the latter difficulty a device consisting of two rings was applied to each insulator of one of the transmission-lines. The top ring was connected to the line wire and the lower one, placed just above the edge of the lower petticoat, was about four times its diameter and was connected to earth. It was also decided to eliminate the defective insulators on the same transmission-line by testing them all to the dry flash-over voltage of 195,000 volts. During the 1909 lightning season it was found that on the line treated only one insulator was disabled, whereas fifty-four were disabled on the duplicate line; the numbers of insulators injured and replaced at convenience were thirteen and thirty-six respectively on the two lines. Only on one occasion was the line treated disabled, whereas the duplicate line was out of commission no less than fifteen times.

H. R. S.

Transmission-Insulators. A. S. Watts.

(Electrical World, New York. 1910, vol. lv, pp. 1294-8.)

The Author compares in detail high-tension insulators of American and European design. The chronological development of American high-tension insulators is described and illustrated, the development from the porcelain head, intermediate shell and glass centre produced 10 years ago to the underhanging insulator, such as the Hewlett and the Duncan type, for voltages of 100,000 and over, being referred to in detail. The latter type has been used at Nortköpping for 500,000 volts. As regards European insulators, the "delta glocke" insulator most extensively used for moderate voltages, the "rillen" type for voltages up to 66,000, the German underhung insulators, particularly one with an enamelled sheet-steel umbrella, are described, and Freise's comparisons of insulating-efficiency of American and European types are quoted. Curves of puncture-voltages of porcelains from European and American materials, and of the relation between test and working electromotive force of American and German insulators, are given, and the Author presents diagrammatically the areas covered by American and German insulators when weight is compared with test-voltages, the figures being compiled from all the insulator-weights available, numbering several hundred. A suggested type of insulator for voltages between 60,000 and 100,000 is illustrated and discussed.

J. A. S.

Dielectric Strength of Oil. H. W. Tobey.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 1171-89.)

This Paper describes an experimental investigation of the insulating qualities of two kinds of oil, namely, medium (specific gravity 0·865 to 0·870) and light (specific gravity 0·845 to 0·850). When free from moisture both these oils have the same dielectric strength—45,000 to 50,000 volts—with a sine-wave, and ½-inch disk-electrodes placed 0·2 inch apart. The factors which affect the disruptive voltage are the shape of the electrodes, the temperature and the moisture. The effect of the shape of the electrodes is illustrated by figures giving the electrostatic fields which were determined by means of magnetic fields obtained by sprinkling iron-filings on to paper placed over the electrodes. The two standard electrodes adopted were ½-inch brass balls fastened to $\frac{3}{16}$-inch rods, the standard distance apart being 0·15 inch, and two 1½-inch brass disks mounted on ⅜-inch rods and placed 0·2 inch apart. Moderate changes of temperature have no great effect, but the dielectric strength is extremely sensitive to moisture, especially in the case of the light oil, and $\frac{3}{100}$ of 1 per cent. of water (namely, about five drops per quart of oil), reduce the dielectric strength to three-quarters. Several methods of testing the oil for dryness are described, as well as the means of drying it. Chemicals can be employed for the purpose, such as calcium chloride, unslacked lime, calcium carbide, etc. The first-named requires 70 hours and the latter nearly 300 hours to dry the oil. A mechanical oil-drier and purifier, consisting of a number of grids carrying blotting-paper, through which the oil is forced, is described. This press will treat 600 gallons of medium oil and 1,200 gallons of light oil per hour.

H. R. S.

Metal-Filament Lamps. J. W. HOWELL.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 819–30.)

The various metal-filament lamps now on the market—especially the tungsten lamp—are here shortly described and compared with the carbon-filament lamp. From experiments made over 20 years ago it was shown that the life of a carbon-lamp varied inversely as the 3·65-power of the candle-power. It was later found that the same law applied to treated carbon-filaments, and it now appears that the same law is also true for metal-filament lamps. A Table of life-factors of tungsten lamps at various watts per candle-power is given, taking 100 as representing the life with 1·25 watt per candle, from which it appears that the factor is 50·65 (that is, the life is nearly one-half) with 1·13 watt per candle and is reduced to 3·03 with 0·75 watt per candle. By use of this Table rapid tests of life can be made in the factory and referred to the standard.

H. R. S.

Resistance to Shock of Metal-Filament Lamps. T. H. MÜLLER.

(Elektrotechnische Zeitschrift, Berlin, 1910, vol. xxxi. pp. 212–3.)

A simple apparatus, used by the Telegraphic Department of the Swiss State Railways at Bern to determine the resistance to shock of metal-filament lamps, consists of a trough 1½ metre long placed at a slope of 1 in 10 on two supports. The lamp to be tested is hung freely, close to the lower extremity of the trough, and an india-rubber ball, weighted with lead, is allowed to roll down the trough and hit the lamp. The weight of the ball is 120 grammes. In beginning a test the ball is started rolling from a distance of 20 centimetres from the lamp, and this distance is increased by increments of 5 centimetres until the filament is broken, the last distance of fall being taken as the measure of resistance to shock of that particular lamp. The results are given of a series of experiments made on a large number of lamps of 16 to 50 candle-power and of 115 to 155 voltage. The average distance of fall which broke the filaments was about 50 centimetres; some few lamps, however, only stood 20 to 30, whereas in others, rupture only took place at 90 or 100 centimetres. There does not appear to be any relation between resistance to shock and watts per candle. The lamps were tested without any current flowing, because when incandescent the filament is so pliable that it cannot be broken; it only becomes deformed after a large number of severe blows. Lamp-globes should be cleaned when the lamp is incandescent.

H. R. S.

Space-Economy of the Single-Phase Series-Motor.

W. S. Franklin and S. S. Seyfert.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 1-18.)

The single-phase motor is larger and heavier than the continuous-current motor of the same rated capacity, principally because the power-factor is less than unity, and on account of the greater core-losses and the power lost in short-circuit currents; there is also a greater tendency to sparking at the commutator. This latter difficulty can be much modified by reducing the frequency, and for this reason a frequency of 15 cycles is preferable for heavy electric traction. The sparking at the commutator can also be reduced by an arrangement of choke-coils, devised by Mr. S. S. Seyfert, so wound that the current can flow readily in one direction but not in the other. With an experimental motor, built on these lines, the results were very satisfactory and no more sparking was in evidence than is usual with the direct-current motor of the same size; a trial without the choke-coils exhibited a superlative degree of sparking.

The design for a new form of single-phase series-motor, having internal fields, rated at 500 HP., 25-cycles and suitable for placing between 62-inch locomotive-drivers, is compared with a design for a similar sized motor having external fields. The losses in the various parts of this motor are given, and it is stated that, by means of this design, the resistance-leads, or other non-sparking device, can be removed from the motor region proper, thus enabling a motor of 60 per cent. greater rating to be placed between the drivers.

H. R. S.

Economical Features of Electric-Motor Applications. C. Robbins.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 587-642.)

The principal object of this Paper is to show how machine-tools can be driven by electric motors with the maximum economy, the data being obtained by actual tests of existing installations. A graphic recording-meter is described—by means of which the load-curve for any machine-tool can be obtained. Several of these load-curves are given as applied to lathes, boring-machines, drills, slotters, etc., from which valuable and interesting information can be obtained. By means of this meter the exact percentage of actual cutting-time can be determined and it is found to average 40 to 50 per cent., but occasionally is as low as 20 per cent. All delays are recorded on the meter, and those which are not necessary can be detected. A record can be made while an expert machinist does the job, thus obtaining a standard for jobs of similar character. In an Appendix curves are given by means of which the power required for various cutting-operations can be obtained, and by means of a series of curves the cutting-speed, the area of cut and the cubic inches of metal removed per minute can be found under varying conditions for drills, lathes, planers, slotters, boring-machines and milling-machines.

H. R. S.

Oil-Switches. F. Dehler.

(Elektrotechnische Zeitschrift, Berlin, 1910, vol. xxxi, pp. 335–40.)

This article describes various new forms of oil-switches (in which the contacts are opened under oil), suitable for high voltages, which have recently been placed on the market in Germany. The principal improvement is that the connections to the cables are through the underside of the box containing the oil, and this has enabled a very compact apparatus to be designed, including electro-magnets for automatically breaking the circuit should the current exceed a predetermined value, and also an ammeter. Several examples of the use of these switches are given. Another form is shown in which the switch can be opened or shut from a distance by means of a solenoid. A description of the application of these switches to the main transformer-stations in Dortmund is given, together with a dimensioned section of the station; there are forty-four oil-switches for 5,000 and 10,000 volts.

H. R. S.

Influence due to Very High Pressures in the Calculation of Lines.

A. R. Garnier.

(Electricien, Paris, 1910, pp. 33–6.)

The Author deals with the question under the following heads:—(1) Choice of pressure; (2) Choice of frequency; (3) Effects of luminescence; (4) Space between conductors; (5) Kelvin effect; (6) Use of aluminium; (7) Self-induction; (8) Mutual induction; (9) Capacity calculations. As regards the first head, an algebraic expression is complicated by variable factors. The Author gives a Table of first approximations, showing the relations between lengths of line in kilometres and the approximate working-pressures used. Under the second head the danger of resonance owing to the natural frequency of long lines is referred to. In the third place the corona effect is treated, a Table connecting maximum voltages allowable, size of conductors and distances between them being given. The spaces between conductors, determined by probability of contact between wires, according to current practice, are given in a Table. By a curve it is shown that the Kelvin effect is negligible for copper conductors under 20 millimetres in diameter. Mathematical expressions are given for the self-induction per phase in two usual arrangements of aerial conductors, and it is pointed out that mutual induction between two lines is a function of their relative dispositions of flux. As regards capacity the subject is treated mathematically, and an expression for the capacity of a circuit in terms of its physical dimensions is obtained.

J. A. S.

Calculation of an Electric Transmission-Line. A. R. Garnier.

(Electricien, Paris, 1910, vol. xxxix, pp. 259–61, 278–9, 300–2, 372–4.)

In these articles formulas of line-economy are first developed, and complete calculations of a transmission-line are then given. The quantities required to be known are the load in kilowatts, the length of line, the power-factor, frequency, number of phases, an estimate of the price received per kilowatt-year, the price per kilogram of the metal conductor, and the interest and depreciation of the line. The unknown quantities are the tension of distribution, the choice of a conductor, the most economical loss, the section of the conductor, the total weight of metal conductors of the line, the total price of such conductors, the interest and annual depreciation, the line-resistance, the Kelvin effect, the calculation of the loss due to the commercial section of wire chosen, the loss in kilowatts and kilovolts produced and delivered, spacing between conductors, capacity, inductive action and natural period of the line, pressure and current at the generating end at full charge, and regulation of the line at cos $\phi = 1$. The whole case is worked out on a 20,000-kilowatt transmission of 150 kilometres length in three-phase energy at 50 frequency with cos $\phi = 0 \cdot 85$, a summary of results being given.

J. A. S.

Brooklyn Line Department: Novel Iron-Bar Overhead Construction.

(Electric Railway Journal, New York, 1910, vol. xxxv, pp. 1016–20.)

Stated to be most interesting and novel is the application on a wide scale of bars in place of wires as more reliable and easier to maintain, and being only installed where traffic is too heavy to be operated at high speeds, they are not hard on the trolly-wheels as might be objected. The adoption of this system, which has been successful, has necessitated the special design of section-insulators, expansion-joints, frogs, diagonals, feed-in castings, and other auxiliaries, most of which are described and illustrated.

C. O. B.

Trackless Electric-Trolley Cars. C. Magnus.

(Teknisk Ugeblad, Christiania, 1910, vol. 57, pp. 282–5.)

The distances within the town of Drammen, Norway, are considerable in proportion to the population, and the question of cost, therefore, entered largely into a scheme for improved communications. After considering several alternatives the system of trackless overhead electric-trolley omnibus service was adopted and opened for traffic in December, 1909. There are 5 miles of double-pole conductors, carried where possible by transverse wires across the street between the houses, otherwise on timber masts with brackets. There are no crossings on the conductors; when cars meet one has to stop and lower its trolley-pole for the other to pass. This has been found to cause no inconvenience, and has the advantage that cars are not bound to certain points for meeting. At termini there are loop lines of 60 feet radius to enable the single-ended cars to proceed in the opposite direction without reversing. The cars can deviate about 13 feet on either side of the conductor-line. The conductors are double insulated against each other and to earth. The two wires are placed 6 inches apart on a common insulator-holder of special design, the span of wires being about 100 feet. The wires are of hard-drawn copper, each of 0·08 square inch area. Three-phase current of 220 volts and 50 periods is supplied by the municipal works at a charge of 95*s.* per HP. per annum. The current is transformed to 550-volt direct current for use on the cars, which are described and illustrated. The total cost of the installation is about £8,000. No trouble has been experienced in running, even on snow in the winter, the cars travelling well over it at very little extra power.

L. S. Z.

Time-Signals by Wireless Telegraphy. L. Rudaux.

(La Nature, Paris, 1910, Premier Semestre, pp. 401–3.)

The great height of the Eiffel Tower at Paris renders it possible to convey messages and signals over a wide range, and in the first installation of wireless telegraphy in 1903 a distance of 1,553 miles was attained with a power below 10 kilowatts. Since that date a provisional military service has been carried on at the Tower, which had the grave disadvantage that no secrecy could be maintained in the despatches, the persons living in the vicinity being able to hear the sound of the sparks. In spite of this, however, the results were so valuable at the time of the fighting in Morocco that the Minister of War decided to establish a permanent laboratory beneath the surface of the Champ-de-Mars, at a distance of about 100 yards from the base of the Tower. The direction of this undertaking was entrusted to the military engineers. Subsequently a joint commission was appointed to decide on the question of the employment of the installation for the despatch of time-signals to vessels and to sea-ports, the proposal being to convey true time from the Paris Observatory to the Eiffel Tower, and to flash mean time thence by wireless telegraphy to all the vessels of the Navy within reach. Two exactly uniform astronomical clocks for this service are installed in a room at the Paris Observatory. The system employed is identical with that in operation at the Besançon Observatory,[1] by means of which uniform time-signals are transmitted to twenty-five manufactories in the vicinity. At the Paris Observatory all the chief clocks, eight in number, have been placed in three glass cases in one room in the basement, in which an absolutely invariable temperature is maintained. The walls of this room are 8·2 feet in thickness. After due precautions have been observed three sets of time-signals are arranged at 12.0 midnight, at 12.2, and at 12.4 A.M. These signals are conveyed by wireless telegraphy over a radius of about 1864 miles, and as the speed of transmission of the waves is about 180,200 miles per second the flash is instantaneous. It is intended to employ shortly more powerful instruments, which will assure a radius of action of 3,355 miles. These accurate time-signals will enable most valuable determinations of longitude to be obtained.

G. R. R.

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[1] Minutes of Proceedings Inst. C.E., vol. clxxvii, p. 381.

Electric Drive in Textile Mills. A. MILMOW.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 437-74.)

A comparison is given of the cost of driving cotton-mills by steam power and by hydro-electric power. The comparative estimates are made out for a mill having 25,000 spindles and requiring about 1,000 HP. It is supposed that the electric power is delivered at 2,300 volts, and the motors are wound for this voltage; no transformers would therefore be needed, except for lighting purposes. It is stated that motors working at this high pressure are quite satisfactory. The capital cost works out to £16 10*s*. per HP. for steam power, and to £7 1*s*. per HP. for electric power. In working out the cost of operation, account is taken of the load-curves for the various days of the week, and it is assumed that the cost of electricity is £5 4*s*. 2*d*. per HP. per year. It is then shown, after allowing for the heating of the mill and other processes requiring heat, that the annual cost of steam power must not exceed £3 12*s*. in order to compare favourably with electric power. It is pointed out that one of the great benefits of electric drive is due to the constant speed attainable, because if there is any variation of speed the maximum must not exceed a certain amount, whereas the output depends on the average speed; therefore the output of a mill can be increased 5 to 10 per cent. by improving the regularity of the drive. A large number of tachograph-diagrams, both for steam and electric power, show the variation of speed in various cases, the improvement in speed regularity with electric power being very noticeable. A curve is given showing the increase in HP. furnished to textile mills by the Southern Power Company, Charlotte, N.C., during the last few years. The supply began in 1904, and had reached 10,000 HP. in January 1907, nearly 40,000 HP. in January 1909, and in January 1910, a little over 65,000 HP.

H. R. S.

Hydro-electric Plants.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 76-163.)

In the course of the discussion on Mr. Doherty's Paper, "Comments on the Development and Operation of Hydro-Electric Plants," [1] Mr. H. C. Short urged the necessity of auxiliary steam-turbine plant, basing his argument on curves he had prepared which gave, for all percentages of the maximum capacity, the cost per kilowatt-hour divided into fixed and operating charges. He then took a typical load-curve for a large city, including lighting, tramways and industrial power for summer, and a similar curve for winter, and obtained therefrom the cost per kilowatt-hour for every hour of the 24 for five different cases; in the first the whole of the power was supplied hydraulically, in the last it was all supplied by steam, and in the other three cases partially by steam and partially by hydraulic power. Finally he derived the result that in the summer the most economical arrangement was to obtain 40 per cent. of the power hydraulically and in the winter 65 per cent. These results depended on the cost of transmission of the hydro-electric power, on certain assumptions of capital expense and depreciation and on the fixed charges which were taken both for the hydraulic and for the steam plant at 11 per cent. Mr. C. T. Hutchinson gave much detailed information as to depreciation for every item and arrived at the following figures:—Hydraulic-power house, 3·4 per cent., equivalent to an average life of 19 years; transmission-line, 2 per cent., equivalent to 26 years; sub-stations, 3·24 per cent., equivalent to 20 years; and he finally worked out that the fixed charges should be reckoned as 17·0 per cent. for steam-turbines and 9 per cent. for hydro-electric power. Mr. Calvert Townley gave 12 and 10 per cent. respectively for these fixed charges. It was stated that a modern steam-turbine plant could be built for £17 per kilowatt of rated capacity, whereas very few hydro-electric plants had cost as little as £30, and generally the cost was as much as £42 to £62 per kilowatt.

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[1] Proceedings American Inst. Electrical Engineers, 1909, vol. xxviii, p. 1513.

H. R. S.

Structural Work at the Tuilière Power-Station.[1]

ALBERT CLAVEILLE.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xlv, pp. 50-200.)

The area above Tuilière drained by the Dordogne and its tributaries is 5,580 square miles, and the river is subject to heavy floods, the flow at Tuilière varying between 8,360 and over a million gallons per second, the water in the latter case, i.e., during an extreme flood, rising 50 feet above its normal level. The river-bed at the point where the dam has been constructed is generally of firm rock, but two pot-holes of clay were encountered requiring special treatment. The dam, which crosses at right-angles, consists of nine masonry piers, forming the guides for eight Stoney sluices, of which the opening of seven is 32·8 feet and of one 23 feet. The piers, which have a width measured transversely to the river of 9·8 feet and a height of 102·7 feet, are founded on a bed of concrete extending from side to side of the river, and toothed into the rock. The top of this bed is at a uniform level of 42·6 feet below that of the impounded water, and betweeen the piers it carries cast-iron sill-pieces against which the sluice-gates butt. Each gate consists of a skin-plating supported by fourteen horizontal latticed girders arranged at distances apart depending on the depth of immersion and the consequent intensity of the pressure; in the live-roller path unequal spacing of rollers is adopted for similar reasons. A double counterpoise for each sluice-gate is carried by "rams-head" blocks, thus, for every foot the gate is lifted, the counterpoise descends 6 inches only. By the use of "gipsy" wheels deriving their motion from electrically-driven winches, the chains which couple to the counterpoises are made to serve also for manœuvring the gates. Eight of these chains, divided into two groups of four, serve for each gate, and the free ends are so attached to an arrangement of rocking-arms as to make each chain take, under all circumstances, its proportion of the load. Adjacent to the dam is the power-house containing nine turbines, the inlet to each of which is controlled by a Stoney sluice, 22 feet wide and 15 feet high. The maximum permissible head of water is 39·4 feet, but the turbines are so designed as to work efficiently at a much lower pressure.

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[1] See following Abstract.

During heavy floods, or in time of extreme drought, water-power is not available, and for use on such occasions a steam-driven installation is provided. The cost, exclusive of machinery, is given as £280,000.

I. C. B.

Generating-Plant at the Tuilière Power-Station.[1]

ALBERT CLAVEILLE.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xlv, pp. 50-200.)

This generating-plant consists of eleven turbo-alternators, nine being driven by water and two by steam. The shafts in all cases are vertical. Each of the hydraulic turbines is furnished with two motor disks keyed on the main shaft one above the other at such distance apart that the water entering through the two sets of guide-blades can be discharged by way of the space thus left. The speed of rotation is 107 revolutions per minute, and the hydraulic governor acting on shutters encircling the guide-blades is so efficient that with a sudden alteration in load of 25 per cent. the speed does not vary more than 3 per cent. from the normal. The turbines can be used whenever the available head of water lies between 19·7 and 39·4 feet, the latter corresponding to the maximum height which can be retained by the dam. Each unit is capable of producing 3,000 HP. with a full head, and the efficiency under such conditions is 75 per cent., dropping to 70 per cent. when working at half load. The steam-turbines are of the Curtis type, and with steam at a pressure of 171 lbs. per square inch, and a temperature of 275° C., the vacuum being 93 per cent., can each give out 3,000 kilowatts when running at a speed of 750 revolutions per minute. The alternators in all cases give out a triphase current of 50 periods at a pressure of 5,500 volts, those used with the hydraulic turbines having fifty-six and those with the steam-driven sets eight poles. The exciters give out a continuous current at 125 volts, and are driven by triphase motors, a single compound tandem steam-engine being available for a first start. The current supplied by the exciters is also used for producing light and power in the station itself and for charging a storage-battery. By means of transformers the tension of the current is raised, and it leaves the station at either 50,000 or 13,500 volts. The distribution is by overhead wires, and the poles which carry the high-tension circuit also provide accommodation for that at the lower pressure and for a telephone service. The high-tension mains served by the Tuilière station have a length of nearly 400 miles and extend to Bordeaux on the west and Angoulême on the north.

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[1] See preceding Abstract.

I. C. B.

Hydro-Electric Power-Station at Montcherand, Switzerland.

P. Schmutz and V. Abrezol.

(Zeitschrift für das Gesamte Turbinenwesen, Munich, 1910, vol. vii, pp. 129–33, 148–51, 168–72, 180–6, and 199–202.)

A detailed account is given of a hydro-electric power-station of 4,000 HP. taking water from the river Orbe. There is a water-conduit, partly in a tunnel cut through the Jura limestone and partly consisting of a reinforced-concrete square-section pipe, 2·02 metres wide and 1·85 metre deep (internal dimensions). In some places this reinforced-concrete conduit is laid on the ground on an ordinary concrete foundation, and small valleys are crossed on reinforced-concrete trestles. The arrangement of the reinforcing-rods is given in sectional drawings of these pipes. This conduit, with a fall of 1·2 per cent., leads into a regulating-reservoir, whence the water is conducted under pressure by a steel pipe 688 metres long and 1·80 metre in diameter. The pipe is divided into five sections of varying slope, each section anchored at one end in concrete, but the pipe itself is exposed, except the last length, which is short and at a considerable slope, and is completely encased in concrete. The turbines have horizontal shafts, and run at a speed of 375 revolutions per minute. There are four units of 2,000 HP. each. Full particulars are given of the turbines, generators and governor-gear.

H. R. S.

Andelsbuch Electricity-Works, Bregenz.

Professor G. Narutowicz.

(Schweizerische Bauzeitung, Zürich, 1910, vol. 55, pp. 15, 33, 62, 79. 52 Figs.)

This is one of the finest hydro-electric power-stations in Switzerland. The Author first describes the large storage-reservoir, in which the waters of the Ache and other streams are collected. The reservoir has an extent of 11·36 acres and a capacity of 7,063,300 cubic feet. From the reservoir two pipe-lines, each 6·56 feet in diameter, are carried down to the turbine-houses, the pipes being of steel plate with riveted joints. Considerable difficulty was met with in the erection, and one of the pipe-lines was seriously injured by a flood which washed out the foundations and supports of the pipes. The power-house at present contains four double Francis-type turbines, each of 2,500 HP. running at 500 revolutions per minute with a normal head of 197 feet of water and passing 141 cubic feet per second. The turbines and accessories are described and illustrated, and each is coupled direct to a three-phase generator, developing 2,250 kilowatts at 5,200 volts and 50 periods per second. The current is transformed up at once to 25,000 volts and fed at this pressure into the overhead line for the distant points, but at 5,200 volts for points nearer the power-house. Great care has been taken to safeguard the high-tension apparatus. The works were begun in 1905 and current was first supplied in 1908.

E. R. D.

Electric-Power Installation at Turifos, Norway.[1] Olav Berge.

(Teknisk Ugeblad, Christiania, 1910, vol. 57, pp. 78–80.)

Electric power is supplied to various mines and for the machinery of the central establishment at distances of 1½ to 10 miles. There will be four power-units (three are already at work), each of 850 HP. The generators are designed for a voltage of 5,000, one for three-phase and two for single-phase alternating-current. They are 12-poled and each is coupled direct to an 850-HP. water-turbine running at 500 revolutions per minute, and has its own magnetizing-dynamo placed on its main axle. The single-phase generators give 480 kilowatts, the three-phase 700-kilowatt alternating-current at 5,000 volts and 50 periods per second, the latter being transformed to 15,000 volts for transmission to the mines. The transformer is of the ordinary core type in oil, with water-cooling by means of gilled pipes and constant flow of about 7 gallons of water per minute. The general arrangement of the installation is shown in a diagram, the apparatus being placed on three floors in a separate part of the power-station. On the ground floor there are four separate transformer-cells and a small repairing-shop. On the first floor, immediately above the transformer-cells, are placed the automatic oil current-breakers, which can be actuated either mechanically by means of hand-wheel and chain, or electrically by two-pole maximum time relays coupled to the transformers on conductors direct from the generators. On this floor is also the switchboard, on which are placed all measuring-instruments, switches, regulators and controllers for the motors of the turbine-governors. On the second floor are the lightning-conductor installation and the arrangement for distributing the current on the long-distance conductors. Of the latter there are two sets, one of 5,000 and one of 15,000 volts, which, at the places where the power is to be utilized, are transformed down to 150 volts for motors and lighting. L. S. Z.

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[1] See following Abstract.

Hydraulic-Power Station at Turifos, Norway.[1] HANS J. HANSEN.

(Teknisk Ugeblad, Christiania, 1910, vol. 57, pp. 66-70.)

The power-water available is derived from the Kobberaa, which in a length of about 3,000 feet has a series of rapids with an aggregate fall of nearly 130 feet, the minimum quantity being about 175 cubic feet per second. The works consist of a dam, conduit and power-station. The dam, which is shown by drawings, is of concrete, about 260 feet long, straight and placed at an angle across the river. The water-level is raised about 35 feet and at flood, when the river carries about 3,600 feet per second, it is calculated that the water running over the top of the dam is about 40 inches deep. The top and back of the dam are faced with granite to prevent injury from the wash. Expansion is provided for by making a break about 5 feet wide, which is closed by needles. Provision is made for carrying timber past the dam on one side, and on the other side is the intake for the power-station. A screen 14 feet wide is placed in front of two sluices, 10 feet in total width, their sills being 12 feet below the top of the dam. From the intake the water is led through a tunnel 1,670 feet long, and a sectional area of 64 square feet, to the regulating-basin, which has a water-area of 1,500 square feet, the walls being carried 7 feet above the top of the dam to prevent overflow in case of sudden stoppage of the turbines. From the basin the water is conducted through a steel tube 78 inches in diameter, $\frac{1}{4}$ inch thick, for a length of 450 feet to an expansion-box, from which to the power-station, a distance of 440 feet, the tube is 75 inches in diameter and $\frac{7}{16}$ inch thick. The tube is supported on concrete piers 18 to 26 feet apart, is anchored in a concrete block at the station end, has angle-rings cast into the concrete at the basin end, and is taken into the power-house at one end, there being branches for four turbines (of which three have been erected) and a through passage for 175 cubic feet of water per second for use in case all turbines are standing. The turbines, calculated to give 850 HP. each at a net fall of 140 feet and 500 revolutions per minute, are of the Francis type, in plate casings, inlets placed eccentrically and regulated by a 40-inch throttle-valve. The governing is effected by bronze leading-vanes, actuated by a hydraulic governor of special construction; safety-valves on the casings are actuated by the governors. The power-plant was tested in August 1908, the maximum power per turbine being 1,150 HP. at 150 feet fall.

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[1] See preceding Abstract.

L. S. Z.

New Sub-stations in Chicago.

(Electrical World, New York, 1910, vol. lv, pp. 459-62.)

The Commonwealth Edison Company of Chicago owns four generating- and thirty-five sub-stations, some of the latter being operated by the railway companies supplying power in bulk direct from the generating-stations to elevated and surface railways, and others, known as "combination" sub-stations, jointly by the railway companies and the Edison Company. Fisk Street generating-station supplies 22,000-volt 25-cycle power generally to underground transmission-lines. Details of the transmission system and of some sub-stations are given. The street-railway service is 600-volt direct-current, private supply for light and power being 2,300 volt 60-cycle. Data of the rotaries, frequency-changers, switch-gear, etc., in the sub-stations are given. The Troy Street sub-station, in the three-wire territory, is also described as an example of a "straight" sub-station to supply alternating-current for lighting and motor service; and an account is given of the means taken to link up the 25-cycle and the 60-cycle systems, the latter having grown to a point where the erection of two 60-cycle 12,000-volt 14,000-kilowatt turbo-generators at the Quarry Street generating-station is necessary. The Indiana Street sub-station, which is of two-storey construction, is also described.

J. A. S.

Hydro-Electric Power-Station at Schaghticoke, U.S.[1]

(La Technique Moderne, Paris, 1910, vol. ii, pp. 193-9.)

This power-station contains four units of 5,000 HP. each. The water-turbines have vertical shafts and are of the Pelton-Francis type. The current is generated at 4,400 volts and transformed up to 32,000 for transmission to Schenectady. The method of testing to obtain the efficiency of these turbines is described and it is stated that great accuracy is attainable. The mean velocity of the water at the entry to the turbine was measured by means of two Pitot tubes, the position of which in the pipe could be varied; one along a horizontal diameter and the other along a vertical diameter. The static head was measured at eight points on the circumference of the pipe. Each Pitot tube and the eight static-pressure tubes were connected to vertical glass tubes mounted on a board, the top ends of which were under equal air-pressure. In order to avoid having to compare each Pitot tube with the eight static-head tubes at every set of observations, a comparison was made once for all and the corrections to be applied when comparing each Pitot tube to the nearest static-head tube were obtained. A complete set of readings for a trial at three-quarter load show that the turbine was developing 3,866 HP. and that its efficiency was 85·26 per cent. The results of several other trials are given, varying between half load and full load. At half load the efficiency was 80 per cent. and at full load 84 per cent.

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[1] *See ante*, p. 313.

H. R. S.

Hydro-Electric Development of the Great Western Power Company, U.S.

(Engineering Record, New York, 1910, vol. 61, pp. 746–8 and 783–5.)

The north fork of the Feather River in Eastern California makes at Big Bend a wide detour of 11 miles, whereas its course, if direct, would be about 3 miles. Through the mountain which formed the obstruction a mining company drove a tunnel 11,768 feet long, and costing over £208,000, to divert the river and thus lay the bed of the detour bare for mining purposes. The gradient of this tunnel, with certain additions affording at its outlet a head of 419 feet, and the failure of the project as a mining operation, have given a Hydro-Electric Power Company an opportunity of utilizing the fall in supplying, by means of a power-plant and transmission-lines, the needs of Oroville, Marysville, Sacramento, Antioch, Oakland and other towns. The work, temporarily complete, will be further carried out in stages increasing with the demand, the dam below the intake to divert and store the water being now of timber 22 feet high, but to be superseded later by a permanent concrete dam 129 feet in height, while several storage sites higher up the river are to be utilized as necessity arises. The tunnel, enlarged and lengthened to a site suitable for dealing with the outfall, is concrete-lined. The turbines, operating at a speed of 400 revolutions per minute, and designed for an ultimate head of 525 feet, developing 18,000 HP., are believed to be the highest-powered in the world.

C. O. B.

McCall Ferry Hydro-Electric Plant on the Susquehanna River.

(Engineering Record, New York, 1910, vol. 61, pp. 684-90.)

This 135,000-HP. station, costing over £2,000,000, being in a large power-consuming area, with the cities of Philadelphia, Baltimore, Wilmington, Harrisburg and Lancaster within a radius of 70 miles, will have a large market. The great dam built across the river, with a length of 2,350 feet, maximum height and width of base 60 and 65 feet, and containing 174,000 cubic yards of 1 : 3 : 5 concrete, creates a lake of about 8 miles long, with a maximum width of 4,000 feet. The principal difficulty is that of dealing effectively with the ice and the extreme fluctuations in the volume of water which is the drainage of 26,800 square miles. For these reasons the dam is a spillway throughout, and a forebay, ice-protection groyne, and other requirements above the entry to the gate-house are provided. The works are described and illustrated, as well as the reconstruction of a long stretch of the Pennsylvania Railway which was interfered with.

C. O. B.

Transmission-Lines of the Central Colorado Power Company.

(Electrical World, New York, 1910, vol. lv, pp. 217-20.)

These transmission-lines are notable because of their length, the size and mountainous character of the territory served, the high voltage used and the severe climatic conditions. Between Shoshone and Denver there are, including a loop to Leadville, 153·4 miles of trunk service, in addition to 28·8 miles between a Denver sub-station and the generating-plant at Boulder. Between Dillon and Denver the line reaches 13,700 feet altitude and wind-velocities of 70 miles per hour are frequent. The single- and double-circuit towers used are described and illustrated, a Table of tests being given. Data of the copper cable-conductors and the galvanized-steel lightning-protection cable are included. Line-sag curves and an illustration of the 100,000-volt suspension-type disk-insulators are given. Though sleet and snow sometimes cover the line-conductors to an outside diameter of 6 inches, and at the top of the Argentine Pass ice and snow on the insulators have frozen them solidly to the towers, there has been no trouble in transmission at 100,000 volts. Static phenomena and arcking at high altitudes in cold weather, when dry snow is blown across the line, are observable, but maintenance of arc does not occur, although power available for this could come from the 10,000-HP. generating-plant at Shoshone and the large steam-plant at Denver.

J. A. S.

Colorado Hydro-Electric Installation.

(Electrical World, New York, 1910, vol. lv, pp. 1649–53.)

This article describes the Glenwood generating-station of the Central Colorado Power Company, the first of several plants feeding a 100,000-volt transmission-line to the large mining camps and cities of Colorado. The station utilizes the regular flow of the Grand River, whose drainage-area above the plant is 4,500 square miles, and a minimum output of about 5,000 kilowatts is obtained. The maximum discharge is in June, the maximum daily discharge noted being 13,358 cubic feet per second. The minimum record (in December, January and February) is 400 cubic feet per second. A limited storage-capacity is effected by a dam, and a 2½-mile tunnel from the intake to the power-house has been driven in the side of the canyon, arranged for a flow of 1,250 cubic feet per second. Two 9-foot penstrokes conduct the water to two 9,000-HP. turbines, each direct-coupled to a 5,000-kilovolt-ampere machine giving a 60-cycle 4,000-volt supply. Particulars of the station-equipment, including the 100,000-volt transformers, are given. An interesting detail, the 100,000-volt roof-bushing or insulated entrance-lead, constituting a variation of the usual methods, is described and illustrated.

J. A. S.

Emergency Generating-Stations for Hydro-Electric Transmission-Plants under Pacific Coast Conditions. A. M. Hunt.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 521–30.)

The hydro-electric power-stations on the Pacific Coast, especially in California, are subject to occasional stoppage for various reasons, such as the failure of the wooden box-flumes conveying the water, and seasons of low water. Standby-stations are therefore necessary, and a comparison is made of such stations when driven by gas-engines or by steam-turbines, the Author endeavouring to prove that the steam-turbine station can be so designed as to be cheaper in first cost, as economical in fuel consumption, with less standby-charges, and can be put into service as promptly as the gas-engine station. The size of station taken for comparison has a capacity of continuous operation of 25,000 kilowatts at 85 per cent. power-factor. In the case of the gas-engine station the twelve units would have a load-capacity of 2,085 kilowatts each at 85 per cent. power-factor, while the steam-turbine station would have two turbine-units of 12,500 kilowatts each at 85 per cent. power-factor. The gas would be produced by means of oil, and in order to get rapidly to work the generators must be kept fairly hot. It is assumed that, with twelve engine-operators, each at his post, it would be possible to get the station into full operation on to the line in 2 minutes.

In the case of the steam-turbine station the boilers would be fired with oil, a small boiler would be kept continuously under steam and an operator would be able to start all the fires simultaneously by turning on the oil and the steam and lighting them by electric ignition. Large thermal storage-tanks would be provided in which the water would be maintained at the steam-pressure temperature by electric heaters energized from the hydro-electric transmission-mains. The energy thus stored would be capable of running the steam-turbines for 30 minutes, that is until the pressure has been got up in the steam-boilers. In this way the steam-turbine station could be got into operation as quickly as the gas-engine station. The comparison of the costs is:—First cost—gas-engine station £27 1*s.* 8*d.* per kilowatt, steam-turbine station £12 10*s.*—Standby-charges per annum and per kilowatt—gas-engines £3 0*s.* 5*d.*, steam-turbines £1 7*s.* 6*d.* The oil-consumption is shown to be practically the same in either case.

H. R. S.

Developed High-Tension Network of a General Power-System.

P. M. Downing.

(Proceedings of the American Institute of Electrical Engineers, New York, 1910, vol. xxix, pp. 531-45.)

This Paper considers the development of the high-tension network in California supplied by hydro-electric power, and its magnitude may be realized from the fact that there are 150 miles of 100,000-volt lines, 1,390 miles of 60,000 volts and 380 miles of 20,000 volts. There are also 1,920 miles of 11,000 volts and under, but they are not included in the network, being regarded as distributing-lines. The paralleling of the stations supplying this network is effected on the high-voltage side, regardless of the length of line between them or of the loads carried. No difficulties have arisen, and the connecting-up is confided to the chief operator, who is called the "load-dispatcher." He is at all times in direct telephonic communication with every part of the system and in his office is a board showing every generating-station, transmission-line and sub-station in the network. The telephone-circuits for this purpose are not run on the transmission-line poles but along entirely different routes. Particulars of the transformers used are given, especially in connection with their insulation, and there are also descriptions of the switches, lightning-arresters and insulators. Trouble has been experienced as regards the latter in the dry season, owing to the salt which is deposited on the surface of the insulators by ocean fogs; the salt is washed off when the heavy rains come.

H. R. S.

Operating Experiences with a Blast-Furnace-Gas Power-Plant.

H. J. FREYN.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 919–1009.)

A full description is given of the average results of daily observations extending over a year of the analysis, of the calorific value and of the purifying of the blast-furnace gas used at the Lackawanna Steel Company's Works at Buffalo. The plant consists of four gas-engines driving alternate-current generators at 83·3 revolutions per minute, of 2,000 kilowatt capacity, 25-cycle, 3-phase, 2,200 volts. Under certain conditions there may be a shortage of gas, reducing the pressure below atmosphere, in which case air would be sucked in and might form an explosive mixture. A serious accident of this nature having occurred, an arrangement was adopted whereby an automatic alarm is given as soon as the pressure of the gas drops below a certain point, so that the necessary steps can be taken, first by cutting off the gas from certain steam-boilers, and if this be not sufficient, by shutting-down one or more of the gas-engines. A chart shows the analysis and the heating-value of the gas for each day during 1909. The maximum calorific value (lower value) determined both by calculation from the analysis and by calorimeter, was 123 B.Th.U. per cubic foot, and the minimum 79, the average being about 100. With a rich gas of 123 B.Th.U. per cubic foot, the engines back-fired and pre-ignited, but when the calorific value was as low as 80 the full output of the generators could not be maintained. Much difficulty was at first experienced in cleaning the gas, i.e., removing the dust. The plant finally adopted for this purpose is described. The first cleaning is effected by a series of zigzag pipes, arranged so that the gas has suddenly to change its direction, thereby depositing the heavier particles of dust. It then passes through wet scrubbers and finally through four Theisen washers, each of 15,000 cubic feet per minute capacity. During 1909, the gas, on reaching the wet scrubber plant contained an average of 1·533 grain of dust per cubic foot, the total weight of this dust for the whole year being 865 tons. The purifying-plant removed 861·7 tons, so that only 3·3 tons entered the gas-engines. The method of determining the amount of dust by means of Brady filters is described in an Appendix. The average thermal efficiency of the engines is 23·77 per cent.

H. R. S.

Electric Power from Blast-Furnace Gases.

RUDOLF POKORNY.

(Stahl und Eisen, Düsseldorf, 1910, pp. 938–44.)

Since it has been proved that the exceptionally heavy work of driving rolling-mills in iron- and steel-works can be efficiently performed by electric power, attention has been directed to the utilizing of the energy contained in the gases from blast-furnaces. The utilization of these gases may be effected in two ways: indirectly by passing them under boilers to raise steam to be used in steam-turbines, or directly in gas-engines. Both gas-engines and turbines may be used directly to produce the blast for the furnaces and power for other work through dynamos, and there is a tendency at present to put these in competition, notwithstanding the known fact that the degree of heat utilized in the gas-engine is, relatively to that obtained by the steam method, as 2 to 1. To obtain all the advantages of the turbine system, the dynamos must generate an alternating current, for reasons set forth in the article. The advantages claimed are: less first cost, smaller space occupied, greater certainty in working, longer life of the plant, simpler construction, less consumption of lubricant and need of fewer men in charge. Hence it is believed that power can be produced by it at a lower cost. The Author shows, however, that the question of space is unimportant, that with purified gases there is no more uncertainty than with steam, and that the cost of power-production is 10 per cent. greater with the turbine than with the gas-engine. The cleansing of the blast-furnace gases is strongly recommended from the economical point of view.

In considering this hitherto little used source of power, the Author bases his calculations on the actual conditions of a large Rhenish ironworks, at which there are five blast-furnaces, each having a daily production of 450 to 500 tons of pig, four being always in blast. Their entire output of 1,900 tons a day is converted into steel and rolled into commercial forms. Electric power is used throughout. The total electric energy required for these works per month amounts to 6,700,000 kilowatt-hours. The volume of the gas issuing from the blast-furnaces is estimated at 340,000 cubic metres per hour. Allowing half of this for heating the blast there remain 170,000 cubic metres for power. Investigations show that of this quantity, when used in gas-engines, 43,000 cubic metres are required for blowing the furnaces and 57,000 cubic metres for power for other purposes in and about the works. This leaves 70,000 cubic metres unused. It is clear, therefore, that all the power required in an ironworks may be derived from the superfluous blast-furnace gases, with often much to spare. The latter may be transmitted to a distance and profitably sold for other purposes.

G. G. A.

Improved Peat-Gas Generator for Power.

(Oesterreichische Zeitschrift für Berg- und Hüttenwesen, Vienna, 1910, pp. 298–302.)

Most of the attempts to utilize the valuable constituents of peat and turf have proved commercially unsuccessful; but fairly satisfactory results have been obtained when the material has been used as a fuel to generate power. In Prussia it is largely used under steam-boilers. The great improvements recently made in suction gas-producing plant, together with the still greater advance in electrical transmission, have again directed attention to the extensive peat-deposits of the world as available sources of power. Peat-gas generators have been brought into practical use but have not made much headway, because the gas produced was not suitable for use in double-acting engines. The chief difficulty with this class of gas-producer, the high proportion of water held in the peat, appears to have been overcome by a Gorlitz firm, which makes what is known as the G. M. A. Peat-gas Generator, of a design extremely simple and certain in action. For use with this generator the firm makes also suitable gas-engines of large size, their one-cylinder, single-acting machines working up to 150 HP., and their one-cylinder, double-acting forms working up to 1,000 HP. With the twin arrangement of four cylinders engines of 4,000 HP. may be built. Within the last few months this firm has erected a complete gas-producing plant and engines of 500 HP. at Jekaterinburg for driving a modern mill, to run continuously day and night through the whole week.

The principal advantages possessed by this plant are economy of heat and a recovery of heat to effect a preliminary drying of the peat, whereby the objectionable excess of water is got rid of. The reducing of the CO_2 to CO requires, as is well known, a relatively high temperature. When the moisture in the peat exceeds a certain percentage in relation to its heating power the whole of the CO_2 is not reduced to CO. Analyses of the gases from the generator at Jekaterinburg, which is fed with wet peat, show very good results in this respect. It appears, therefore, that the system is applicable under ordinary conditions wherever there is an abundant supply of peat. In the Author's opinion a plant erected on a peat-moor for the generation of electric current to be transmitted to a distance would be commercially successful.

G. G. A.

Shear-Strength and Elasticity of Concrete. M. Grübler.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1909, vol. liii, pp. 449–55.)

Previous experiments made to determine the resistance to shear of materials have given results which are so much at variance that further research was considered necessary. The Author contends that the actual value of the shear-stress is not of very great importance but that it would be of far greater interest to experiment on various bodies made in the same way and of identical materials, and to subject them to various kinds of shear-stresses, in order to ascertain whether the coefficient of resistance found experimentally agrees with the theoretical value. The test-pieces consisted of hollow concrete (one part cement and three parts ordinary sand) cylinders of 23·7 inches external and 7·9 inches internal diameter and 3·95 inches high. The results of these tests are shown in Tables and diagrams. C. J. G.

Grain-Pressures on Silo-Walls. E. Lufft.

(Beton und Eisen, Berlin, 1910, vol ix, pp. 76–9.)

After due consideration of the difficulties entailed in the accurate measurement of the pressures of grain on the walls of the silo, the Author was led to believe that the best solution of the problem might be obtained by dealing with the pressures exerted by the column of wheat in the same way as with the corrresponding pressures observed in a column of liquid or gas, using a manometer for the purpose. For these experiments it was necessary to substitute for a portion of the wall of the silo a thin membrane which should separate the grain from the fluid employed, which latter might then convey to the mercurial manometer the pressures caused by the superincumbent contents of the silo. He therefore made use of a shallow, circular, flat cast-iron vessel filled with water and embedded in the silo-wall. This was covered with a film of india-rubber, inserted flush with the stucco rendering of the circular brick wall of the silo-shaft. From this water-chamber two tubes were passed through the wall of the silo, the one communicating direct with the manometer, while the other, which was turned upwards, was provided with a stop-cock, to permit of the escape of the air when the water-chamber was filled. The manometer was one of the usual type, with the scale divided into millimetres, by means of which direct readings could be taken. In one set of experiments the manometer was replaced by an ordinary hydrometer, such as is employed for indicating the water-level in tanks. In this latter case it was deemed advisable to calibrate the hydrometer beforehand by comparison with a column of mercury. A thermometer was likewise used to admit of temperature corrections. Diagrams explain the arrangement of the apparatus, and precautions were taken to adjust the readings to allow for the outward pressure on the rubber film due to the water in the chamber. Two sets of silos, each circular in section, served for experiments, one 23·87 feet in diameter and the other 11·28 feet in diameter, and each 54·77 feet in height. The larger silos held about 540 metric tons and the smaller 120 tons of wheat. Graphic diagrams represent the increased pressures at equal intervals of time during the filling operation, and the pressure-lines are regular curves; the readings were taken in some cases at intervals of 3 minutes, in others every quarter hour. The curves are almost uniform in character when the filling is regular, and with dry wheat, weighing 64 lbs. per bushel, good diagrams were obtained, with a steady increase of pressure, but if the filling were interrupted or stopped the pressure immediately became relaxed and the contents of the silo clearly became consolidated. With dirty or damp grain the curves differ, and the smaller silos gave a flatter curve than the larger ones. The Author states that the experiments prove that wheat in bulk partakes most of the properties of a fluid; that when at rest the bulk becomes consolidated, and it gradually assumes more of the nature of a solid body, and thus exerts less pressure on the silo-walls; and lastly, that in the emptying of the silo the grain again manifests fluid properties, which increase the faster the operation is conducted.

G. R. R.

Strength of Cast-Iron Punch- and Riveter-Frames.

A. L. JENKINS.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 723–44.)

Experiments were made to determine the strength of cast-iron frames for punching- and riveting-machines. Tensile and cross-breaking specimens were cast on to the frames and the tests were made with the Riéhle machine. Altogether eighteen frames were tested of different shapes, some plain and the others having flanges, and photo reproductions are given showing the fractures. The results of the tests were compared with the usual beam formulas, with the Resal formula and with the Pearson-Andrews formula, and it was found that these three formulas gave results 14, 43 and 75 per cent. in excess of the actual strength of the castings. None of these formulas are therefore satisfactory, and there appears to be no definite relation between the strength of the curved cast-iron beam and the transverse strength of the test-bar cast with it. The mathematics of the subject are discussed in an Appendix.

H. R. S.

Stresses and Strains in Elastic Expansion-Piping.

Professor A. BANTLIN.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 43–9.)

The object of the tests, carried out at the Technical High School, Stuttgart, under the supervision of the Verein deutscher Ingenieure, was to find a relation between the forces exerted on expansion-piping and the stresses set up in the material of which the piping was made. Tests were made with mild-steel and cast-iron pipes, the results being tabulated and compared with the theoretical values. The equations for calculating the strain in curved rods when subjected to stress were found to agree with the actual results obtained on test both for solid mild-steel horseshoe-shaped test-pieces and cast-iron horseshoe-shaped expansion-pipes. The bending-stress for such pieces may therefore be calculated by means of the formulas used for curved rods. On the other hand it was found that for mild-steel piping these formulas gave results far below the measured deflections. The main cause of this apparent anomaly is due to the folds which are formed on the compression-side of the pipe when expansion of the piping takes place. The cast-iron expansion-piping showed no such deviation. The stress on mild-steel expansion-joints is far smaller than would be gathered from the deflection of the pipe, but greater than the calculated value obtained from the bending-formula used for curved rods. This calculated stress in the pipe cannot correspond with the actual stress experimentally obtained, since the shape of the pipe differs from the one assumed in the formula.

C. J. G.

Tests of Stamping-Machines by Cylindrical Compression Test-Pieces. H. GAUSSLEN.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 882–6.)

Except by means of hydraulic presses there is apparently no way of testing stamping- and compression-machines. This defect gives rise to commercial irregularities and can only be remedied by some simple method of testing. In addition, owing to lack of data the design of these machines is based on guess-work, resulting frequently in the production of machines incapable of performing the work for which they were intended. The Author describes a simple method of employing annealed copper cylinders, compressing these under the stamping-machine and determining the latter's power from the amount and rate of compression of the cylinder. Electrolytic copper is used in order to ensure purity and uniformity. Since the compression per inch-length of test-piece varies with the ratio of the height of the cylinder to the diameter, the Author made several experiments in order to arrive at the most suitable ratio. The results of these tests are reproduced graphically. He recommends for practical purposes a cylinder 0·0844 inch high and 0·76 inch in diameter, giving a ratio of 0·111. If compressed to 0·0668 inch the force exerted by the machine, as obtained from the curve, is 36,520 lbs. For greater pressures the number of cylinders used may be increased. In conclusion, he shows how to calculate the power required to emboss a plate of tin to a prescribed compression, and in solving the problem illustrates the usefulness of the method employed and also the utility of curves prepared specially for this purpose, stating that the results thus obtained might not be absolutely accurate, but certainly tended to obviate the irregularities referred to. C. J. G.

Essential Problems of Aviation. HECTOR POULEUR.

(Revue Universelle des Mines, Liége, 1910, vol. xxix, pp. 1–77.)

The Author discusses both theoretically and practically the conditions of aviation as at present understood. The first problem to be considered in designing a heavier-than-air machine, monoplane or biplane, is its lifting-power. This is dependent on the form and dimensions of the "wings" and the angle at which they are set to meet the wind, i.e., the angle of incidence of the air-current, supposing the air to be moving and the plane at rest. The under or bearing surface of the wings may be straight or curved, the latter form being generally adopted. Experiments show the common assumption that the upper surface contributes nothing to the lifting-power is not justified. A complete set of experiments carried out by Mr. Rateau has enabled him to formulate the conditions of flight in precise mathematical terms. He has shown that the air-pressures are proportional to the square of the velocity, at least within the limits of 7 to 35 metres per second. This confirms previous experience, so that the law $F = c\,SV^2$, in which formula F represents the upward thrust or lifting-power given by the surface S acted on by an air-current of V velocity, may be taken as definitely proved. For a cylindrically curved wing, the coefficient c becomes $c = K\,(a + a_0)$, a being the angle of incidence and a_0 the angle at which the upward thrust is nil. Putting $a + a_0 = \delta$, the angle of deviation or deflection, which differs from a by an angle a_0, variable with different profiles of surface and nil with a straight wing, the general formula for the lifting-power $F_v = KSV^2\,\delta$ is obtained. The coefficient K is the same for all surfaces having the same dimensions, but it varies with the form, and increases in a notable degree with the lateral extension or spread. Thus for a plane or wing 500 × 300, K = 0·18, while for another 500 × 150, K = 0·275. As an example the Author calculates by this formula the lifting-power of a cylindrically curved wing 500 × 300 millimetres set to offer an angle of incidence of 10° to a wind of 20-metres-per-second velocity. To an angle of incidence of 10°, corresponds an angle of deviation δ of 14° 30′. Whence $\delta = 2\pi\,\dfrac{14 \cdot 30}{360} = 0 \cdot 252$. And $F_v = 0 \cdot 18 \times 0 \cdot 5 \times 0 \cdot 3 \times 20^2 \times 0 \cdot 252 = 2 \cdot 72$ kilograms. The lifting-power, therefore, is about 18 kilograms per square metre.

The formula for the horizontal thrust, which for an aeroplane is one of the essential terms of resistance to forward motion and therefore a factor in the speed of flight, is $F_h = KSV^2\left(\dfrac{\delta^2}{2} + ma + n\right)$,

a being the angle of incidence and m and n two constants dependent on the surface. It is important to bear in mind that this resistance increases very rapidly after a certain angle is reached, and that consequently lifting-power must not be sought by too great an angle, especially as the power begins to diminish at about 20°.

The Author, after briefly discussing the relative merits of the monoplane and the biplane, considers the question of stability, coming to the conclusion that the future lies with a cellular form and the tailed type. The chief dimensions and other characteristic features of the several types of aeroplanes in actual use are given.

G. G. A.

Problem of Flight. J. G. Lepper.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 314-16.)

Continuing the experiments of Mr. A. Ph. Kapteyn,[1] the Author carried out an investigation on the partial vacuum ("depression") created at the top of the plane of a flying-machine. A gauge resembling the well-known water-gauge in principle was used. It was found that in order to attain the most favourable depression on the upper side of the plane, the front edge of the carrying-planes should form an angle of 7° to 8° with the horizon and gradually diminish towards the tail end of the plane to 0° (i.e., horizontal). The best curve appears to be a parabola, since this shape offers the least resistance to the tail stream-lines.

H. C. A. T.

Present Knowledge of the Theory and Practice of Aeronautics.

F. Bendemann.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 786-91, 851-8, 888-96 and 933-9.)

Section 1 of the Paper deals with the historical development of the airship, special reference being made to the experiments of Otto Lilienthal. Section 2 relates the theory of flight in which: (*a*) it is stated that gliding is made possible only by the generation of air-currents; (*b*) the lifting-power of the propeller is determined theoretically, enabling the efficiency of a machine to be readily estimated; and (*c*) close investigations into the effects of air-resistance indicate the most suitable shape to be given to the planes of a flying-machine. Section 3 deals with experimental investigations on air-resistance. Special stress is laid on the value of experiments with models, and reasons are given why the latter cannot entirely replace experiments on a large scale. A new device for studying gliding is described, and the results obtained from an experimental installation are discussed. Section 4 deals with the state of practical knowledge on the subject up to the end of 1909. The Author concludes that the improvements to be made consist in rendering the aeroplane immune from the wind-action, especially when rising or landing. The most important flights of the year 1909 are fully discussed with a view to indicating the lines on which improvements should be made, the various systems being critically compared. C. J. G.

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[1] Minutes of Proceedings Inst. C.E., vol. clxxix, pp. 464-5.

Geodesy and Aeronautics. Dr. Max Gasser.

(Zeitschrift fur Vermessungswesen, Stuttgart, 1910, vol. xxxix. pp. 410–14.)

The Author states that, as the airship and the aeroplane become more perfect, means will have to be found to determine accurately their position relatively to the earth's surface. In order to locate position in a horizontal plane, the Author proposes to use a sextant possessing an artificial horizon such as is obtained with the gyro-horizon sextant of Admiral Fleuriais, which is accurate to within a few minutes. In a mist the airship's position may be obtained by making use of the fact that the magnetic dip varies for different parts of the earth's surface. This difference, for example, amounts to 5° between Kiel and Friedrichshafen. The double magnetic needle of Dr. Bidling would be useful in this connection, especially since this instrument is not affected by the rolling or pitching of the airship. The necessity of some such instrument is emphasized by the fact that owing to a sea-mist Blériot at one time lost his bearings when crossing the Channel. The distance travelled is very difficult to determine in a fog owing to the lack of an instrument corresponding to the nautical log. The velocity may, in fine weather, be determined by mean of a Bertelmeyer compass, the method of using the same being carefully explained.

C. J. G.

Critical Survey of Existing Flying-Machines.

Ausbert Vorreiter.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1909, vol. liii, pp. 1093–1102 and 1140–8.)

This is an account of the present achievements of the aeroplane, a discussion of the theory of flight and a description of the various machines in use. The Author concludes with the following summary:—

1. The bi-plane is more stable than the mono-plane, so that for the same carrying-capacity the former need not have so great a

width as the latter. Its construction lends itself very well to the stiffening of the structure. The disadvantages of the bi-plane are:—greater weight and greater air-resistance for the same carrying-capacity, demanding increased motor-power and working-expenses.

2. Automatic stability cannot be depended on in gusts of wind (side winds). It is of the utmost importance to keep the two sides in equilibrium, this being best effected by bending the side planes; pivoted planes are useless. Only one plane should be bent, viz., the one on the inner side of the curve, since otherwise it would be necessary to use the side steering-gear. It is of special advantage to be able to bend the horizontal steering-plane as well as the bearing-planes.

3. It is best to place the horizontal and vertical steering-planes behind the bearing- or carrying-planes, also to place the latter in the imaginary line connecting the propeller with the centre of gravity of the machine.

4. Two propellers revolving in opposite directions should be adopted, in line with each other axially to prevent tilting of the machine should a propeller break. Since the second propeller would now be placed in the air-current generated by the first, a special arrangement has to be made so that each is supplied with the same amount of air. The propellers may be driven either by a differential gear or direct.

5. Air-cooled cylinders should only be used for small powers and then always with a revolving cylinder-casing, otherwise the motor would be insufficiently cooled. Motors with revolving cylinders have the additional advantage that the rotating mass of the cylinders adds to the automatic stability of the machine.

6. All steering operations should be effected by one gear only, either of the lever or the wheel type.

7. Some means should exist for allowing the motor to run free so that the gliding motion of the machine is not impeded.

8. The machines should be able to start either with a spurt obtained by a short run, or by means of some arrangement such as that used by the Brothers Wright.

9. Bi-planes and mono-planes seem to be the most successful.

10. Increased flying-velocity is to be encouraged in order to reduce the size of the planes.

11. Wood is recommended for the skeleton frame, and cloth impregnated with rubber for the carrying- and steering-planes.

C. J. G.

Evolution of the Aeroplane.

(Revue de l'Aviation, Paris, 1910, vol. v, pp. 76–80 and 98–101.)

Owing to the rapid progress made with aerial locomotion, many machines produced only two years ago are already considered fantastic. It seemed therefore advisable by means of a series of photographs to depict some of the chief stages in the evolution of contrivances for flying which are heavier than air. Santos Dumont, the first who flew in France, was incited to make the attempt by the account of what had been accomplished by the Brothers Wright. His original apparatus had the wings inclined upwards at a rather steep angle, so as to ensure lateral stability, and he made use of vertical partitions or screens instead of ailerons. The longitudinal stability was somewhat precarious, as was the case also with the American machines, but his first model was on good lines. However, he did not persevere with this, as he considered that canvas was unreliable, and he evolved an aeroplane constructed entirely of wood. This also suffered owing to defective longitudinal stability, and he reverted to the employment of canvas. Ultimately he came upon the "dragon-fly" type and aimed at excessive lightness of build. With a machine of small size he used a 40-HP motor, and he lost his life in attempting flight on the self-same day as Delagrange. Gabriel Voisin, though not the actual inventor, was the creator of the biplane type of machine, and was the first to make use of the appliances which, as modified by Henri Farman, Curtiss, Maurice Farman and Roger Sommer, have made the aeroplane available for all aviators. Voisin and Bleriot, who had different ideas, worked together, and could not readily come to a mutual understanding. The first model biplane belonged to Delagrange and was piloted by Charles Voisin. The central compartment had no vertical partitions, and a slight upward tilt of the wings was adopted in order to maintain the lateral stability of the machine. The changes effected by Farman are discussed, and out of them arose the modified biplane machine of Delagrange. After many alterations Voisin returned almost to his original model. Reference is made to the collaboration between Gabriel Voisin and Colliex and to the construction of the abortive *Flying Fish.* The machine introduced by H. Farman is based on the biplane of G. Voisin, but Farman for a time dreamed of a triplane. It is asserted that Blériot had great difficulties at the outset in obtaining adequate longitudinal stability for his monoplane and was diverted for a time from his own plans by his admiration for the machine of the Brothers Wright. His first monoplane capable of flight was produced in July, 1907, and for more than a year he was retarded by failure to maintain stability. His essays in various types of machines are indicated up to his model No. 12. In contradistinction to the many changes made in the apparatus of Blériot, Levavasseur effected relatively few alterations. The Gastambide-Mengin machines are described and illustrated, and Latham's Channel-flight is discussed. It is said that the most remarkable point of the machine he employs is that he alone can use it successfully. The biplane-apparatus of Sommer is stated to be one manifesting signs of real progress.

G. R. R.

Longitudinal Equilibrium and Curvature of Bearing-Surfaces of Aeroplanes. R. Arnoux.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 46–59.)

The bearing-surfaces of all planes present to the air a concavity to increase their power of sustentation. This advantage is, however, largely compensated by instability of longitudinal equilibrium. The conditions of dynamic equilibrium are explained and a parallel is drawn between planes provided with surfaces of different forms and the beam of a balance. Jœssel in 1870 reduced to the formula $d = (0 \cdot 2 + 0 \cdot 3 \sin i)\, l$ a law discovered by the Abbé Avanzini of the pressure of water on a flat surface. Here d = distance of point of application of resultant pressure from edge of attack, l = depth of plane, i = angle of incidence of currents. Following up the investigation the Author has produced a diagram showing the position of centre of pressure for various angles of attack on surfaces of various degrees of curvature. If a curved surface in equilibrium with a small angle of attack be acted on by a disturbing force, the centre of reaction is carried out of the vertical through the centre of gravity, and a couple is formed which destroys equilibrium. If the centre of pressure be considered as a function of the angle of attack, the sign and value of the angular coefficient which the tangent to the curve in the Author's diagram makes with the axis of the angle of attack, decides whether the equilibrium is stable, neutral, or unstable. The slight displacement of centre of pressure for small angles of attack, makes the employment of a supplementary reaction necessary. This is provided by additional surfaces, whose angular disposition must be such that the attacking angle of the first surface to meet the air must be greater than that of the surface which follows. The flat surface with a dynamic equilibrium, always stable, has, however, the disadvantage of inferior power of sustentation to the curved surface. A Table shows for angles of attack of 3° to 10° the values of the reactions of the air, normal and parallel to its direction, and the ratio of these reactions for flat surfaces, and surfaces curved $\frac{1}{8}$, $\frac{1}{12}$, $\frac{1}{16}$, $\frac{1}{20}$, and $\frac{1}{30}$. The high values of the ratio of normal to parallel reaction for plates curved $\frac{1}{30}$ justify the adoption of this form in the latest apparatus.

W. H. B.

Siemens-Schuckert Aeroplane. O. Krell.

(Deutsche Zeitschrift für Luftschiffahrt, Berlin, 1910, vol. xiv, pp. 13–5.)

After due consideration of the question of design the Siemens-Schuckert factory decided to proceed at first with a machine of the bi-plane type, which appeared to have been the more successful at the time the experiment was begun. An examination of the

French models led to the determination to build a machine of considerable size, the planes having an extreme spread of 54·12 feet, with an area of about 646 square feet; the weight was 1,212 lbs. The trials of the new apparatus by the constructor, Mr. J. J. Bourcart, took place on the plain of Bornstedt, near Berlin, where a shed had been erected, with the consent of the military authorities, and the experiments began on the 31st December, 1909. In the first instance the tests were limited to rolling forward on the wheels at a speed of about 23 feet per second, but later, when a speed of 26 feet per second was attained the machine rose from the ground some 32 feet in consequence of a puff of wind, and was carried a distance of about 393 feet, when, the engine having been stopped, it glided back to the ground slightly injuring the extremity of the left-wing plane. The experiences gained by this involuntary flight involved a few slight changes in the apparatus: the tail was doubled in length and some small stabilizing planes was added to it. The ends of the wings were curved upwards and some alteration was made in the steering gear. On the 9th March, 1910, further trials were effected, and on the 11th of that month, when a somewhat stiff breeze was blowing, the aeroplane, with three aviators on board, was brought out for further rolling experiments. The machine, however, rose from the ground, and, after a flight in a straight line of some 270 yards, described a sharp curve, and with a considerable inclination amounting to some 35° struck the ground, all the aviators being thrown out and more or less injured. The machine itself was wrecked, but the 50-HP. Argus motor was practically unharmed.

G. R. R.

Internal-Combustion Motors for Aeroplanes and Dirigible Balloons. CH. DANTIN.

(Le Génie Civil, Paris, 1909, vol. lv, pp. 111-54.)

Although the motors required for aeronautics are in principle similar to those used for road-cars, the design is, in many cases, totally different, while even where the general form of the motor-car-engine is adhered to, important modifications in detail are found necessary. The conditions which a motor for use with a dirigible balloon must fulfil are by no means the same as those which must be complied with by the engine of an aeroplane; for instance, in the former case extreme lightness is by no means so necessary, nor, considering that the supporting platform is not likely to incline very far from its horizontal position, is the same difficulty encountered in securing a uniform supply of fuel to the cylinders or an efficient lubrication of the moving parts. In the case of the motor for a dirigible balloon, however, a danger which does not occur with the aeroplane, has to be guarded against, viz., that arising from the possible ignition of gas escaping from the envelope.

To secure extreme lightness special steels are used for the cylinders as well as for connecting-rods, crank-shafts, etc., while owing to the machine being used in an atmosphere practically free from dust, the casings may be dispensed with if thought desirable. Aluminium combustion-chambers have been tried and, where water-jackets are employed, the same metal has been used but the different coefficient of expansion of the latter metal as compared with steel, militates against the use of such composite structures. Twenty-two different types of motor are described more or less in detail, among these being various designs of the star form, in which the cylinders are spaced radially round a circle at equal distances apart. In one of the latter types the crank-shaft remains fixed while the rest of the machine rotates, the object being: (1) to secure more efficient air-cooling; and (2) to make use of the weight of the cylinders themselves for the purpose of a fly-wheel. In order to avoid undue vibration the minimum number of cylinders is four, while a larger number is more usual. The weights of the motors per horse-power are exceedingly low, 3½ lbs. to 6 lbs. being common, while in one case 2·1 lbs. per horse-power has been reached. Two designs are shown where the same valve serves both for inlet and exhaust, but the introduction of this modification would not appear altogether to make for simplicity. I. C. B.

Rotary Motor for Aviation Purposes. Prof. M. Varille.

(Revue de l'Aviation, Paris, 1910, vol. v, pp. 140–1.)

Some years ago a motor suitable for burning heavy oils was placed on the market and enabled a saving to be made of 75 per cent. as compared with the cost of employing petroleum-spirit. This motor, when used for wagons of 3 to 5 tons, or for taxi-cabs, has given good results, and the same company have now introduced a new type of rotary motor adapted for aeroplanes. The total weight of a machine indicating 40 HP. is 330·6 lbs. It is thus not an extra-light motor but is suitable for industrial purposes. Its ordinary speed is 900 to 1,000 revolutions of the cylinders, and its minimum and absolutely constant rating is 40 HP. It is an eight-cylinder engine, the diameter of the cylinders being 3·73 inches, and the stroke 5·1 inches. The possible number of revolutions may vary between 200 and 1,100 of the cylinders, or 400 and 2,200 of the cranks. A change of speed by gearing, arranged with ball-bearings to minimize friction, placed on the frame of the motor, enables the speed of the propeller to be reduced by about 1,000 to 1,100 revolutions per minute. Like all the Burlat motors the present one relies on the theorem of La Hire. The cylinders are diametrically opposed in groups of two and two, and the pistons of each group, thus constituted, are united by rigid rods. The crank-shaft, constructed like that of a four-cylinder motor, constitutes, with the pistons, the cylinders and the bed-frame, a powerful fly-wheel which serves to steady the running and tends to safeguard the screw-propellers. This motor works on petroleum-spirit with magneto-ignition. The rotation of the cylinders secures very perfect air-cooling, and the fact that the cylinders are arranged in four different planes allows each a free track. Lubrication is effected by a pump, the feed of which is capable of adjustment, and the centrifugal action keeps the cylinders and bearings well oiled.

G. R. R.

Installation of an Aerodynamic Laboratory. G. Eiffel.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. i, pp. 7–44.)

The Author describes an installation for finding the resultant pressure in magnitude and point of application on any surface, the movement of the air about the surface, and the front and back pressures. The air is drawn from a large shed into the testing-room through a pipe in which it is given uniform direction and velocity. The velocity of the air may be deduced from the formula $V^2 = \frac{2\,gh}{\delta}$, where δ = the specific weight. A comparative table of the results so attained, and the velocities registered by the Recknagel and the Casartelli anemometers, and also by the Pitot tube, decided the Author to accept the results of the last-named method. The pressures on a surface exposed to the air-current are measured by an aerodynamic balance. Calculations are given for an example in general terms and for a particular case, in which the coefficient of total resistance is found to be 0·0785, and the horizontal and vertical components 0·017 and 0·076 respectively. The plate is curved, with a versed sine of 0·42 inch; its dimensions are 35·37 by 5·9 inches, and its chord makes an angle of 15° with the horizontal. A description is given of the method adopted to arrive at the distribution of pressures, of a method to determine directly the centre of pressure without the aid of the aerodynamic balance, and of the direction of the air-currents about a plate inclined at 40°, at 80°, and normal to the wind. The plates on which the Author has carried out tests are:—(1) curved with a versed sine of 0·42 inch, dimensions 35·37 inches by 5·9 inches: (2) flat 33·40 inches by 5·9 inches. For each of these is given, in tabular form for various angles, the distance of the centre of pressure from the edge of attack, the resultant pressure for a wind of 32·8 feet per second, the coefficients of total resistance, the vertical and horizontal components and their ratio, the angle between the resultant and the vertical, and the angle between the resultant and the normal to the plate or chord. Diagrams of unit-pressure and of position of centre of pressure are also given in each case. By superimposing the two diagrams of unit-pressures, the Author demonstrates the superior sustentation of the curved plate, especially at slight inclinations, justifying the preference of most aviators for a curved-bearing surface.

Diagrams of curves of equal pressures on concave and convex surfaces, and of pressures on the median section, are given for angles of 10°, 15°, 20°, 90° inclination to the direction of the wind in the case of the curved plate, with remarks on the results, followed by a Table showing the mean front and back pressures, total pressure, and ratios of the pressures. Similar data are given for the flat plate, the angles being 5°, 10°, 20°, 60° and 90°. From these it appears that on the curved plate the front pressure is one-third of the total up to 20°, while on the flat plate it is one-fifth. The force of the air on the plate is therefore largely due to the back pressure for small angles. The mean front, back and total pressures on both plates for inclinations to the direction of the wind of 5°, 10°, 15°, 20°, 60°, and 90° are given in tabular form.

The Paper concludes with a diagram of a curved plate inclined at 15°, showing that though the angle of attack be zero the air-currents are inflected on arriving near the edge.

W. H. B.

Dirigible Passenger-Balloon "Deutschland."[1]

HEINRICH PUDOR.

(Deutsche Zeitschrift für Luftschiffahrt, Berlin, 1910, vol. xiv, Heft xiii, pp. 16-17.)

The passenger-balloon *Deutschland*, registered as L.Z. VII., was the largest balloon in the world. Its extreme length was 485·5 feet and its cubic capacity 671,000 cubic feet. The cabin, fitted up after the manner of a railway saloon-car and approximately of the same size, was situated midway between the two cars and communicated with them by footways. The ribs were formed of aluminium, but were in all external parts cased with mahogany. As is usual in railway-saloons, there was a passage down the centre, with seats right and left consisting of ornamental wicker-work arm-chairs. The floor, composed of five thicknesses of wood veneer, was covered with carpet, and the ceiling, carried by eight ornamental columns, was inlaid with mother-of-pearl. Seats were provided for sixteen passengers and the crew was composed of two engineers, three fitters, two captains and a guard (Führer) in charge stationed in the front car. When fully charged the benzine-tanks carried a supply sufficient to run all three motors for 15 hours. The motors, furnished by the Daimler Company, were each rated at 140 HP. It was estimated that the speed reached would amount to 37·28 miles per hour. The window-openings in the cabin reached only half-way down the sides, and there were no apertures for observation in the ceiling. The cabin, slung from the airship by twelve steel wires, was provided with lavatory accommodation and a restaurant.

G. R. R.

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[1] This balloon was destroyed on its second voyage.

Destruction of the Delitzsch Balloon by Lightning.

F. BAUER.

(Deutsche Zeitschrift für Luftschiffahrt, Berlin, 1910, vol. xiv, Heft ix, pp. 16–20.)

The Delitzsch balloon ascended from the works, Elektron II., at Bitterfeld at 6.10 p.m. on Saturday, the 16th April, 1910. It was inflated with hydrogen gas, and carried four passengers; a voyage lasting until Sunday evening was contemplated. The balloon had a capacity of about 44,500 cubic feet, was 47·5 feet in diameter and weighed 893 lbs. complete. It carried about 1,350 lbs. of ballast in forty-one bags. On the next morning at 8.0 a.m. the news was received that the wreck of the balloon with the corpses of the four passengers had been found at 5.30 a.m. at the village of Reichensachsen, about 96 miles distant, in the Kassel district. The ruins of the car were caught in an apple tree, and the general features of the country surrounding the spot where the accident occurred are explained by reference to a sketch map. The village is situated in a valley basin near the edge of the Eschwege plain, and according to the testimony of the cottagers there was a violent thunderstorm during the night, which began about 12.30 a.m. and lasted till 1.15 a.m. There was not much wind, but heavy showers of rain and hail with vivid lightning and thunder. The watches of all the occupants of the car had stopped at 12.58 a.m., and there was distinct evidence that the balloon had been struck by lightning which passed upwards through the body of Mr. K. Luft, who was in charge. By reference to a series of photographs of the boots, and by means of markings on the body, the direction of the current was ascertained. The Mannesmann steel tube, used to form the ring of the car, was perforated by the passage of the electric fluid in several places, photographs of which are appended. From the ring the electricity ascended into the body of the balloon, and a diagram shows that, while in the lower part the hydrogen gas had been displaced by air, there was between the pure gas in the upper part of the balloon and the air below an explosive mixture of air and hydrogen, occupying the central zone. The flash ignited this mixture and caused a violent explosion which burst the balloon.

G. R. R.

Military Auto-Motor Search-lights. LUCIEN FOURNIER.

(La Nature, Paris, 1910, Deuxième Semestre, pp. 8–11.)

Search-lights for army purposes have hitherto been drawn by four or six horses on special vehicles prepared for the purpose, but since the use of motor-cars has become prevalent it appeared advisable to provide automobiles for this apparatus. A photo-electric service has been equipped, the motors being supplied by the De Dion-

Bouton Company and the electric apparatus by Bréguet. The car differs little from those in general use, and consists of a chassis with a platform in lieu of a carriage-body. The 18-HP. motor can maintain a speed of 18·6 miles per hour. The sole difference between this vehicle and the ordinary car is in the use of a dynamo direct-coupled to the motor, which furnishes a current of 110 amperes at a potential of 90 volts, with 1,250 revolutions per minute. The search-light with all the apparatus for its manipulation is mounted on a four-wheeled truck, quite independent of the automobile, which latter serves solely the purpose of transporting it from place to place. When the vehicle reaches its destination, the search-light truck is lowered from the platform of the car by means of an inclined plane and block and tackle. A flexible cable, which has in normal cases a length of 328 feet, unites the dynamo and the search-light. The manipulating apparatus is also quite independent of the photo-electric plant, and the man who works it remains near the car. All the steering is thus done at a distance, and should the electric apparatus fail to act, the search-light can be moved by hand. The search-light is in the form of a cylinder provided at the back with a metallic parabolic mirror, 35½ inches in diameter. The light is furnished by an arc-lamp placed before the lens; it consumes 110 amperes at a potential of 50 to 54 volts. In front of the cylinder is a screen composed of vertical shutters, operated by a rack driven by a special electric motor. The search-light is thus adapted for optical telegraphy. Electro-motors are employed to impart to the search-light movements in a vertical or a horizontal direction, so as to enable the cone of rays to sweep the horizon or to be elevated or depressed. There are thus three electro-motors for the above motions and for working the shutter. The portable manipulating apparatus is enclosed in an aluminium box, provided with an arrangement of straps and braces, so that the operator has complete freedom of movement.

G. R. R.

Promotion of Safety in the Transport of Explosives in the United States. Colonel B. W. Dunn.

(Journal of the Franklin Institute, Philadelphia, 1910, vol. clxix, pp. 167-201.)

A bureau for the safe transport of explosives and other dangerous articles—created as the direct result of an explosion of a car-load of dynamite on the Pennsylvania Railway near Harrisburg in May, 1905—began active operations in June, 1907; in August, 1907, the membership had increased to 173 companies in the United States and Canada, operating 222,629 miles of line. The work is carried out by twenty-one inspectors, whose duty is to examine methods of warehousing and packing explosives for transport and to make periodical reports. The manufacturers of explosives are Associate Members of the bureau, attend conferences and discuss all proposed rules and amendments. Before a car can be used for transporting the more dangerous explosives it has to be duly examined, both by a railway employee and by the shipper or his agent, printed certificates detailing the points examined have to be signed by each, and to be attached to the side of the car. A complete list is given of the principal inflammable articles and acids, stating the maximum quantity allowed to be packed in one parcel, as well as the flash-point of the inflammable articles. Facsimiles of numerous labels used, and photo-reproductions of the effect of explosions which have taken place on railways, are given.

H. R. S.

Oscillograph. J. A. Johnson.

(Transactions of the Canadian Society of Civil Engineers, Montreal, 1909, vol. xxiii, pp. 208-17.)

The oscillograph has rendered it possible to record graphically alternating and potential wave forms. The Author describes the two general types of instruments based on the two types of galvanometer, (1) the movable coil type, and (2) the movable magnet type. The difference between the ordinary galvanometer and the oscillograph is that the natural period of vibration of the latter is very short; it may be as small as $\frac{1}{50,000}$ second. In the stationary coil or vibrating-strip type of oscillograph, the moving element consists of a strip of iron tightly stretched and placed between the poles of a powerful electro-magnet. The centre of the strip is thus magnetized crosswise. Stationary coils are placed before and behind the strip, through which the current to be investigated passes. The angular deflection of the strip is made to record by means of a mirror reflecting a ray of light. The vibrating loop type of instrument is virtually a D'Arsonval galvanometer of very high period. The moving element consists of a single loop or turn of conductor, the two sides of the loop being tightly stretched between the poles of a strong electro-magnet. The current to be investigated is passed through the loop, and the deflection is recorded by means of a mirror. To give good results an oscillograph should have (1) A short, free periodic time compared to the period of the wave forms being recorded; (2) Critical damping; (3) Negligible self-induction; (4) Sufficient sensibility. Photographs show alternating wave forms taken from a 12,000-HP. generator of the Ontario Power Company of Niagara Falls, the charging current of a 60,000-volt transmission-line and the corresponding generator-potential, and the effect of a short-circuit of a 12,000-HP. 12,000-volt generator at three-quarters voltage.

R. S. B.

Water-Seismograph. G. Lippmann.

(Comptes Rendus de l'Académie des Sciences, Paris, 1910, vol. cl, pp. 363-6.)

This form of seismograph consists of a tube having a cistern at each end, the whole being filled with water; it is in effect a water-level. If the equilibrium be disturbed the liquid oscillates, and the period depends only on the dimensions of the apparatus, and it is shown that it is equal to

$$2\pi\sqrt{\frac{l}{g}\cdot\frac{S}{2s}},$$

where l is the length of the tube, S the cross-section of each cistern, and s the cross-section of the tube. If, for instance, the length of the tube be 1 metre, the area of the tube 4 square centimetres, and the area of each of the cisterns 400 square centimetres, the period of oscillation is 14·14 seconds, and if the area of each of the cisterns be increased to 1 square metre the period becomes 68 seconds. It is therefore easy to construct such an apparatus so that its own period will be far greater than the period of the seismic movements—a condition which is not always easily filled with the ordinary horizontal pendulum. The motion of the water is communicated to a thin mica disk placed in the tube and acting as a piston, and connected to a mirror by means of which a photographic record is obtained.

H. R. S.

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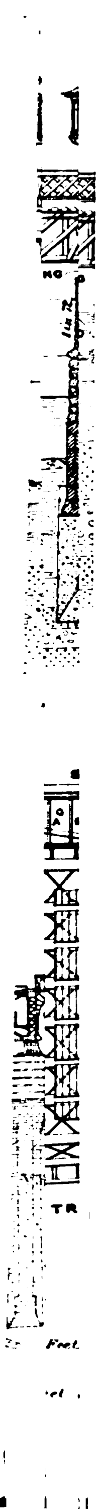

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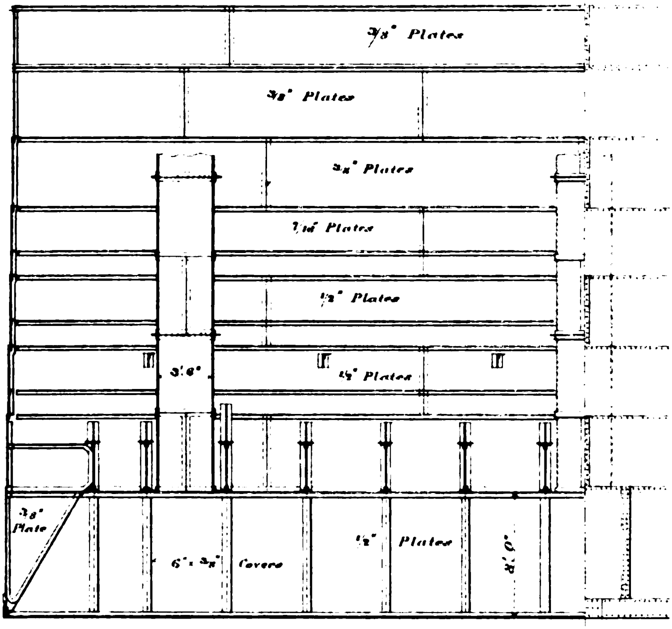
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⅝" Plates
9/16" Plates
½" Plates
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½" Plates
⅜" Plate
6" x ⅜" Covers
½" Plates
8'0"

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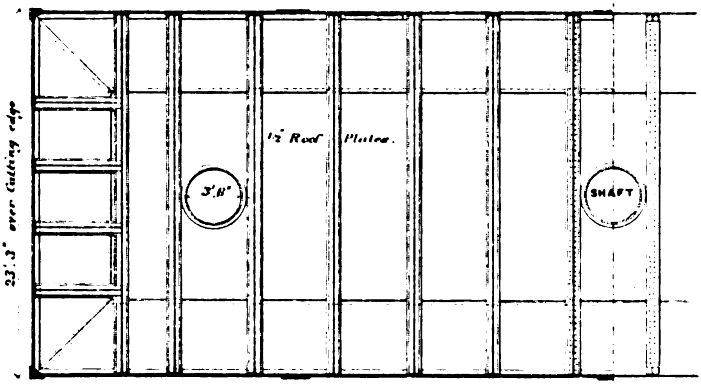
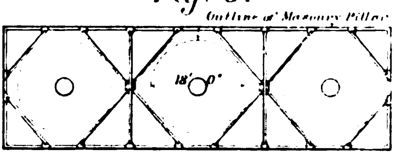

23'3" over Cutting edge
½" Roof Plates.
3'8"
SHAFT

HALF SECTIONAL PLAN.

DETAILS OF CAISSON C 1.

Fig: 8a.


Outline of Masonry Pillar
18'0"

PLAN
SHOWING BRACING.

THOS KELL & SON, LITH. 40 KING ST COVENT GARDEN

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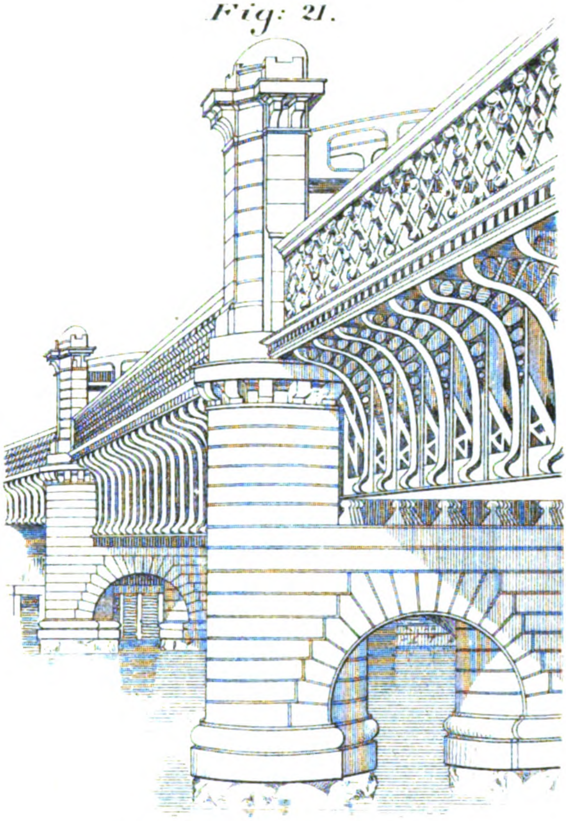
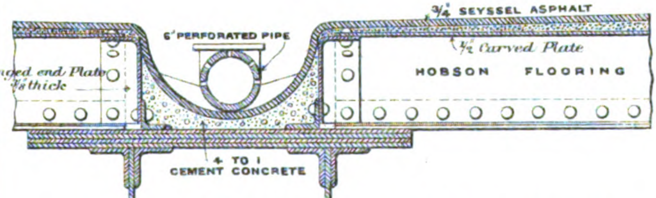
Fig: 21.

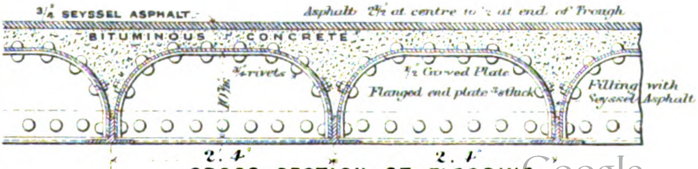


VIEW OF BRIDGE.

Figs 20.


¾" SEYSSEL ASPHALT
6" PERFORATED PIPE
½" Carved Plate
HOBSON FLOORING
Flanged end Plate
⅜ thick
4 TO 1
CEMENT CONCRETE

SECTION SHOWING DRAINAGE AT MAIN GIRDERS.


¾" SEYSSEL ASPHALT
Asphalt 2½" at centre to ½" at end of Trough
BITUMINOUS CONCRETE
¾" rivets
10⅞"
½" Carved Plate
Flanged end plate ⅜" thick
Filling with
Seyssel Asphalt
2'.4"
2'.4"

CROSS SECTION OF FLOORING

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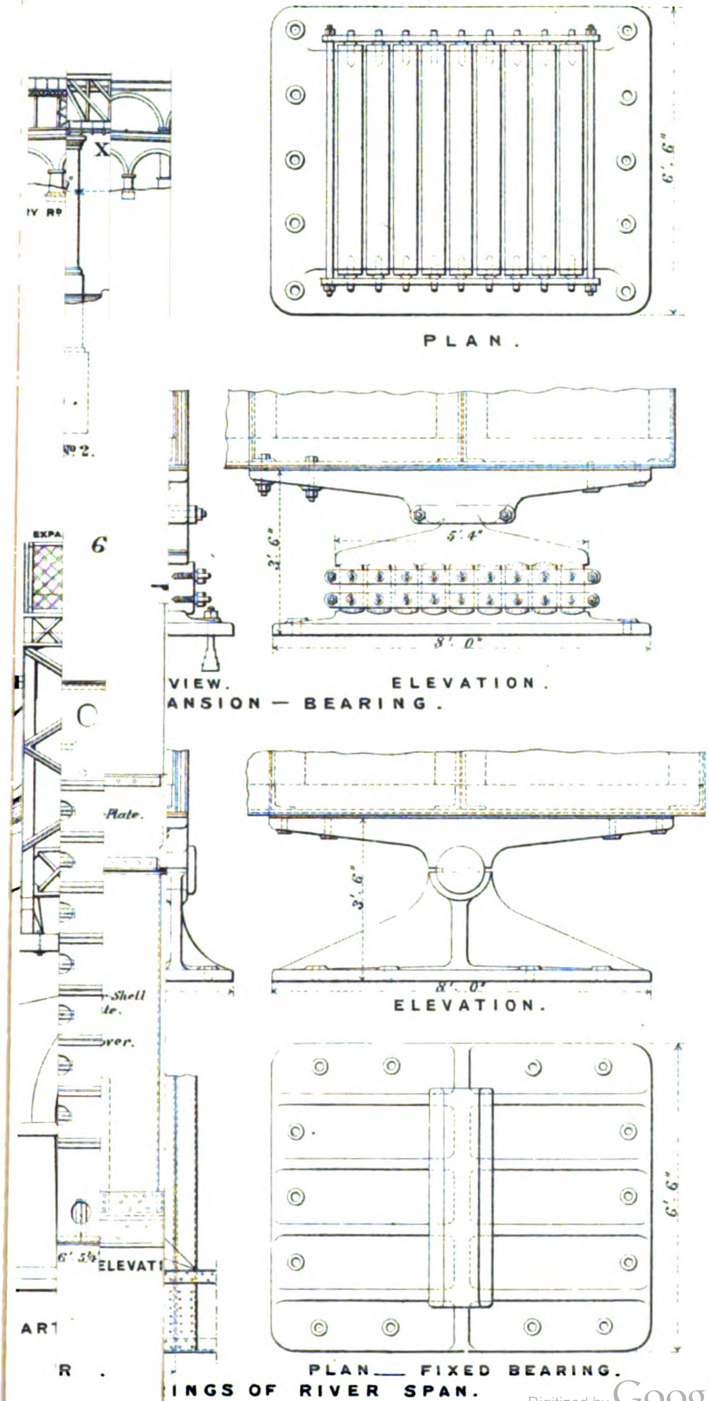
Figs 9.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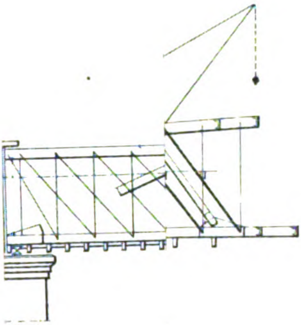
6' 6"
PLAN.
2' 6"
5' 4"
8' 0"
VIEW.
ELEVATION.
ANSION — BEARING.
3' 6"
8' 0"
ELEVATION.
6' 6"
PLAN — FIXED BEARING.
INGS OF RIVER SPAN.

ST-BL



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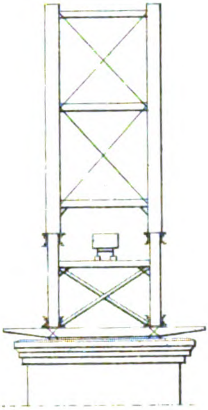

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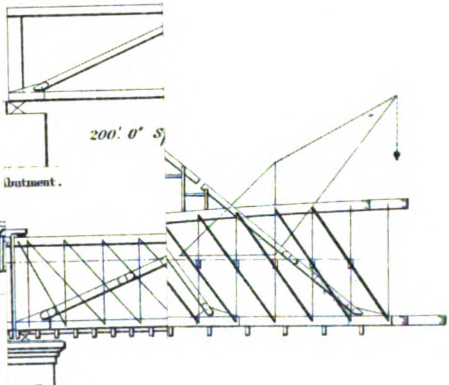
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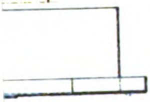
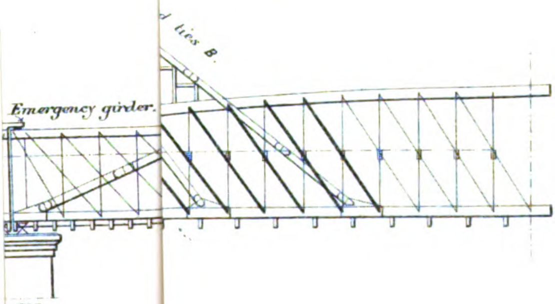


NOTE

The Ties indicated by thick lines were stiffened to resist compression during the erection.


d ties B.
Emergency girder.

No 3.

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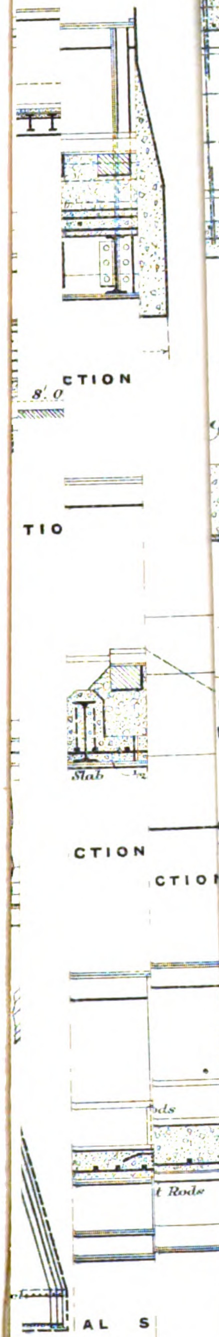
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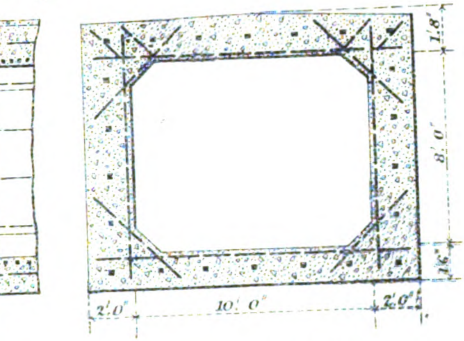
Figs 10.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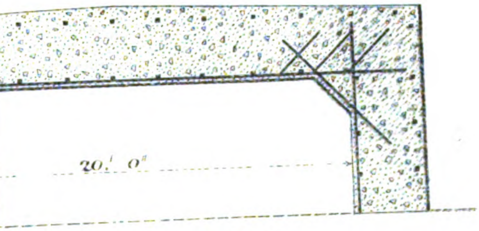
10.' 0"
2.' 0"

CROSS SECTION

FOOT CULVERT

gs 11.



LF CROSS SECTION

D 20-FOOT CULVERT

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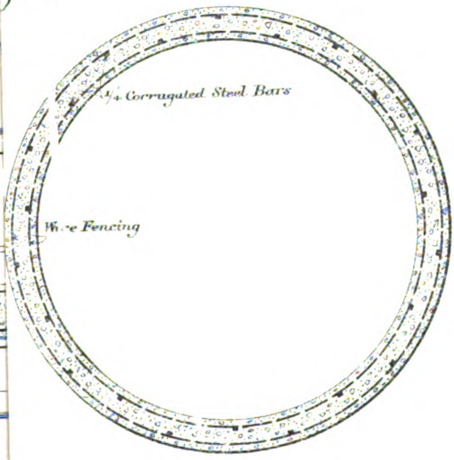
¾ Corrugated Steel Bars
Wire Fencing

CROSS SECTION

G RING OUT E: 4-FOOT SIZE

rt IV.

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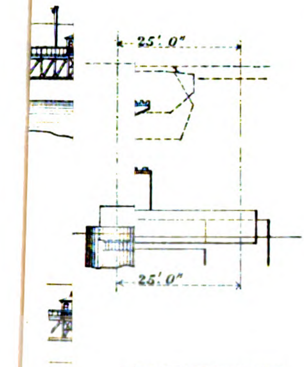

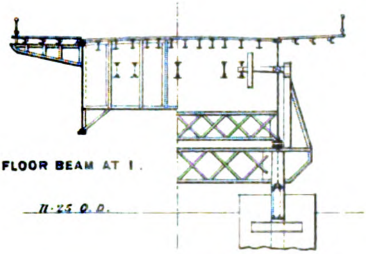
FLOOR BEAM AT 1.
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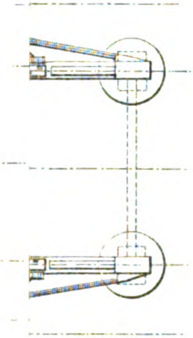
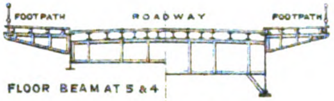
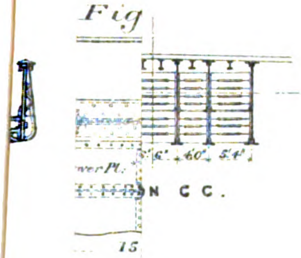
SECTION AT B B.




FOOTPATH
ROADWAY
FOOTPATH
FLOOR BEAM AT 5 & 4

FLOOR BEAM AT 3 & 2.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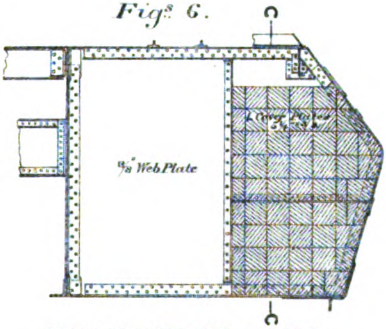
Figs 6.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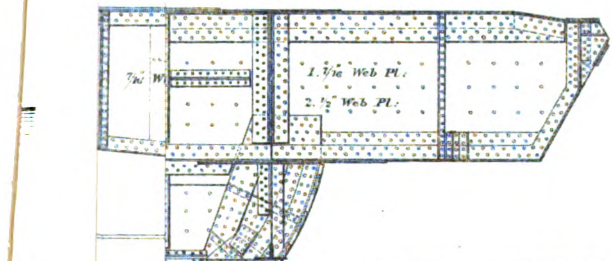
⅞" Web Plate
Cover Plates

COUNTERWEIGHT GIRDER.


7/16" W
1. 7/16" Web Pl.
2. ½" Web Pl.

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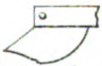
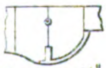
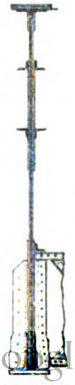


SHAPE OF ½ W. PL.

¾" SIDE PL.



SHAPE OF 7/16" WEB PLATE.



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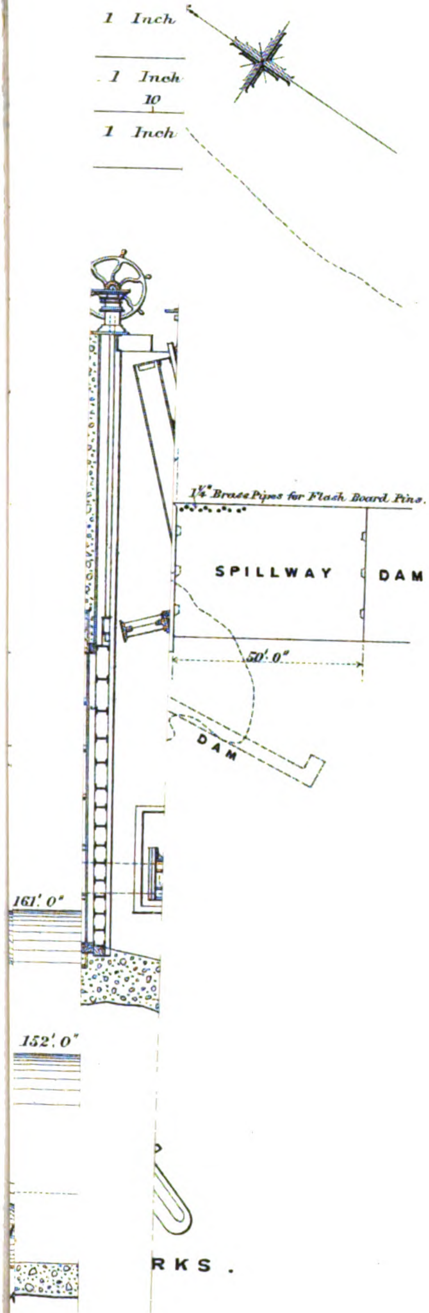
1¼" Brass Pipes for Flash Board Pins.
SPILLWAY
DAM
50' 0"
DAM
161' 0"
152' 0"

RKS .

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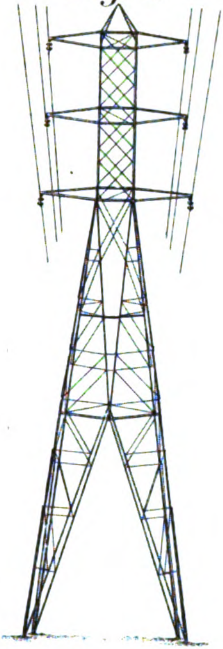
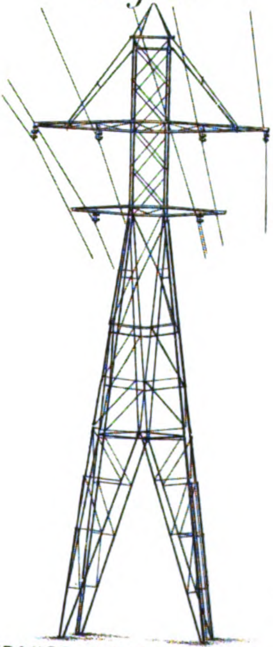
Fig: 5.



STANDARD TOWER.

Fig: 6.



TRANSPOSITION TOWER.

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